Study of the effect of the curing protocol, the RAP source, and the compaction method on cold recycled mixes with foam mechanical characteristics

by

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Etude de l'effet du protocole de durcissement, de la source rap et de la méthode de compactage sur des mélanges recyclés à froid aux caractéristiques mécaniques de la mousse

Amir RAHMANBEIKI

RÉSUMÉ

Les mélanges froids ont fleuri au cours des dernières décennies et ont contribué à de nombreuses constructions de chaussées. Les avantages environnementaux et économiques justifient tous les investissements et efforts scientifiques pour développer cette méthode. Cependant, la complexité du comportement des mélanges froids limite son application. En raison de ses caractéristiques et de ses performances, de la forte dépendance à l'ambiance et de nombreux facteurs en phase de pré-production, de production et de post-production, un contrôle de qualité est nécessaire au cours des différentes étapes de la production. D'autre part, l'utilisation de la mousse de bitume comme liant y ajoute encore plus de complexité, ce qui nécessite un processus de construction plus minutieux. Bien que la stratégie d'utilisation du RAP dans la structure des mélanges froids aide à fournir des aspects de durabilité plus qu'auparavant, les différentes caractéristiques du RAP et de la matière première nécessitent une compréhension plus récente. En raison de toute cette sophistication associée à la production de mélanges froids, des normes uniformes et des protocoles de production sont difficiles à établir et sont donc rares à trouver. Le durcissement, en tant que l'une des principales étapes de production efficaces, se révèle crucial et étudié dans cette étude. Les caractéristiques finales et les propriétés techniques sont liées à cette partie du procédé et c'est principalement en raison du comportement évolutif des mélanges froids. De plus, les spécimens réels extraits sur le terrain présentent des caractéristiques différentes de celles du laboratoire et témoignent de la difficulté de la simulation des conditions de terrain en laboratoire. Dans la rédaction actuelle, cette partie d'un processus de production de mélanges froids est étudiée et propose une méthode de durcissement pour simuler les conditions réalistes sur le terrain et minimiser l'écart existant entre les résultats des tests en laboratoire et sur le terrain afin d'améliorer la réflexion en conditions réelles dans le laboratoire. Différentes sources de RAP et différents processus de compactage sont également étudiés pour éclairer davantage le problème. L'application de la nouvelle méthode de durcissement dans cette étude a abouti à des spécimens plus rigides qui sont 3 fois plus rigides que les spécimens avec durcissement conventionnel. La résistance à l'humidité est également améliorée et les résultats de l'ITR sont modifiés en moyenne de 10% dans la plupart des cas. La résistance à la rupture testée par le test SCB est également modifiée. Cependant, ces changements dans les résultats SCB, ITS et ITSM ne sont pas toujours cohérents.

Mots-clés : Recyclage à froid; mousse d'asphalte; durcissement; RAP, compactage.

Study of the effect of the curing protocol, the RAP source, and the compaction method on cold recycled mixes with foam mechanical characteristics

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ABSTRACT

Cold mixtures have been burgeoning in the last decades and assisted in many pavement constructions. Environmental and economical advantageous justify all the investment and scientific efforts to develop this method. However, behaviour complexity of cold mixtures restricts its application. Due to its characteristics and performance high dependency to ambience and numerous factors in pre-production, production and post-production phase, quality control is necessary during different stages of production. On the other hand, utilization of foam bitumen as binder casts even more complexity on it, which requires a more meticulous construction process. Although, RAP usage strategy in cold mixtures' structure helps to provide sustainability aspects more than before, however, the different characteristics of RAP and raw material requires newer understanding. Due to all this sophistication associated with cold mixtures production, uniform standards and production protocols are hard to establish, so are rare to find. Curing, as one of the main efficacious production stages, is found to be crucial and investigated in this study. Final characteristics and engineering properties are tied to this part of the process and it is mainly because of the evolutive behaviour of the cold mixtures. Moreover, actual field-extracted specimens yield different characteristics than laboratory fashioned ones and it is a witness of hardship of field condition simulation in the lab. In present writing, this part of a cold mixtures production process is investigated, and it proposes a curing method to simulate the realistic field condition and minimize the existing gap between lab and field specimen test results to improve real condition reflection in the laboratory. Different RAP sources and different compaction processes are investigated as well to cast more lights on the issue. Applying the new curing method in this study resulted in stiffer specimens which are up to 2 times stiffer than the specimens with conventional curing. Moisture resistance is also improved and ITR results are, on average, changed by 10% in most of the cases. Fracture resistances, which are tested through SCB test, are also changed. However, these changes in SCB, ITS and ITSM result are not always consistent.

Keywords: Cold recycling; foam asphalt; curing; RAP, compaction.

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LIST OF ABBREVIATIONS

AC	Asphalt Concrete
BSM	Bitumen Stabilized Material
CAM	Hot Mix Asphalt
CBTM	Cement Bitumen Treated Material
CIR	Cold In-place Recycling
CM	Cold Mixture
CR	Cold Recycling
CRM	Cold Recycling Mixture
CTM	Cement Treated Material
FDR	Full Depth Reclamation
FSD	Free Surface Drying
HIR	Hot In-place Recycling
HMA	Hot Mix Asphalt
PC	Partially Confined
RA	Reclaimed Asphalt
RAP	Reclaimed Asphalt Pavement
SGC	Shear Gyratory Compactor
VA	Virgin Aggregate

INTRODUCTION

Roads have always been an inseparable part of the civilized world and date back to more than 4000 years B.C. Development and roads necessitate each other in the modern world. Humankind started to find new habitats and to do so, roads seemed essential. Today, there are tons of kilometres of roads all over the world including Canada. In Canada, there are 1,042,300 kilometres of roads so far and this number is an ever-increasing figure due to population growth, and advances in equipment and knowledge makes the development happen in a faster way than before.

Every progress need resources, and in road construction, they are fuel and raw material. Since this massive usage of raw material took place in a very long term, now we are facing with their scarcity. The need for such material and its rareness promote sustainability discussion. Sustainability is a simple concept and a complex one at the same time. The most comprehensive definition is provided by the Brundtland Report in 1987 (Imperatives, 1987) which states: *"Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs."* Moreover, with the 1970's oil crisis, companies and other executives decided to get independent of oil and its price (Godenzoni, 2017). To do so, many researches and practical efforts were conducted or funded by them which broke new grounds. As a result, projects cost, environmentally and financially, declined since new decisions and policies started to be applied. The road industry, as a big material and oil consumer, has been affected by sustainability enormously. New equipment and new executive approaches emerged as the outcomes of many years' efforts revolving around sustainability. Material specification, hauling and type beside design protocols and standards have been undergoing lots of changes and revisions.

Bituminous layers, as the very first layer of the road which touches tensions, has a crucial role to play in a matter of performance and durability of useful life of a road section. However, this role is not fulfilled without interaction with the core of the road structure. In other words, a

desirable performance of a flexible pavement, is a result of a synergy between the bituminous layer and sub layers. As it comes to the rareness of qualified raw material, reusing the existing old pavement particles seems to be the only solution. Reclaimed Asphalt Pavement (RAP) comes from milling or scraping old pavement and after processing, it can be used in a new pavement.

Formerly, Hot Mix Asphalt (HMA) was the only solution which widely used in the structure of roads. However, innovations make it possible to reach almost the same performance criteria as HMA in an easier and cheaper way, financially and environmentally. Cold mixtures came up as a competent rival for HMA since they use many times less energy and leave a smaller environmental footprint. New equipment capable of changing bitumen shape to emulsion or foam which are easier to haul and apply, make it possible to use bitumen in the structure of the pavement without heating.

In the 1970s, foam mix was introduced as one brilliant technique to substitute hot mix which uses way more energy and fuel. However, complex behaviour in different conditions makes it difficult to have those material widely recognized by road engineers. Different reactions of foamed mixes to different humidity, temperature and traffic conditions required a better understanding of its nature. To do so, quality control of material in the pre-production phase, fresh state, curing, and performance of these mixes has been under the radar of many scientific centres. However, these complexities cause lots of limitations in establishing acceptable standards which predict and judge the performance of these mixes.

Specimens for material characterization and behaviour analysis come in two ways. They either are cored out of the road body which has undergone traffic and aging or, are manufacturing specimens in laboratory conditions. In most of the cases, the results of these two kinds of specimens are inconsistent which means lack of site simulation techniques in the laboratories. The laboratory specimens are manufactured and tested in more controlled conditions. Moreover, field specimens experienced different curing conditions and traffics, even thought they would come from the same road.

Many scientific endeavours have been carried out to minimize the inconsistency between laboratory made and field made specimens test results. Meanwhile, curing, due to its complexity, is one of the hardest problems to address. The specimens cored out of the field have experienced a long-term curing and it is not feasible in the laboratory to provide such exact conditions due to cost and time constraints. Consequently, curing needs to be simulated in a shorter time period which makes the consistency, between laboratory and field results, harder to reach. This thesis tries to discuss curing and proposes a curing method which hypothetically is believed to reduce the field and laboratory results inconsistency. Moreover, different RAP sources and compaction effects are evaluated to investigate it in a more comprehensive perspective.

CHAPTER 1

RESEARCH PROBLEMS AND OBJECTIVES

In this chapter, asphalt pavement rehabilitation is firstly defined, and it is followed by a short explanation of different rehabilitation techniques. The pavement recycling is covered and different techniques for recycling are listed and explained briefly.

1.1 Asphalt Pavement Rehabilitation and Recycling

Deficiencies in technical processes, shortage of good quality material, lack of good quality control and so forth, can result in serious damages such as moisture damages, potholes, ravelling, cracking and rutting in the early ages of service life of the flexible pavement. Therefore, rehabilitation and maintenance are inevitable courses of actions in order to prolong the service life of the pavement and skip big expenses of reconstruction. On the other hand, the importance of sustainable approaches, in which the raw material and fuel consumption are prioritized peculiarly, is a well-established fact. Development in equipment and process over many years, made hot asphalt mixtures unrivalled pavement and rehabilitation technique. However, due to sustainability concerns and application requisites in some cases, there has always been a need for a competent method to compete with traditional solutions. Due to a long history of the road and pavement development, designers are faced with the lack of virgin aggregate and raw material. On the other hand, the dependency of the HMA and other heat-using techniques on fuel price, and their bad environmental effects decreased their popularity; howsoever, they keep their dominance yet. These two propel the subsequent scientific and innovative attempts to generate an alternative approach in a matter of pavement rehabilitation. In summary, cold recycling technology deserves further investigations due to their development potential and market demands (Xiao, Yao, Wang, Li, & Amirkhanian, 2018).

Pavement requires evaluation and maintenance over time. Due to lots of cyclic loading and variable ambience conditions over time and along the road, rehabilitation is a necessary measure to keep pavement in a desirable shape, and acceptable condition in terms of

functionality. To this end, three rehabilitation techniques emerged out of this context. Asphalt overlaying, reconstruction and recycling, Figure 1.1.

Asphalt overlay technique is basically covering the existing pavement with a new layer of asphalt concrete, which adds an extra strength to the previous layers of the pavement and provides a new surface for riding. Before laying the fresh asphalt concrete, the deficiencies in the existing pavement, like potholes and cracks, must be removed or repaired (Thenoux, Gonzalez, & Dowling, 2007). Sometimes asphalt concrete deterioration before placing a new asphalt layer is required. In these cases, removing the stripped material is not necessary since it does not affect the results and it imposes unnecessary costs (Johnson & Freeman, 2002).

Replacing the whole structure of an existing pavement, including granular and base layers, with a new structure of the pavement is called reconstruction. Reconstruction needs demolishing and gathering the whole material in a central plant. After that, designing and construction of different layers of the new pavement start. Throw-away-and-star-again seems a good term which implies reconstruction. It is mostly used when rehabilitation is mixed with upgrading exercises which results in significant changes of the road alignments (Wirtgen, 2012).

Recycling is one of the popular rehabilitation methods due to (Oke, 2011; Xiao et al., 2018):

- Its cost effectiveness;
- Less environmental impact;
- Less fuel and raw material consumption;
- Less curing time;
- Safety;
- Easiness of the application and potential pavement improvements;
- Not disturbing subgrade material;
- Eliminating disposal problem.

Recycling allows us to use up to 100% of RAP in new pavement structure (M Bocci, Canestrari, Grilli, Pasquini, & Lioi, 2010; Paige-Green & Ware, 2006; Santagata & Chiappinelli, 2003). Meanwhile, using foamed bitumen as the binder in the structure of the new mix is a very popular solution. To do so, after pulverizing the distressed layers, foamed bitumen, cement and limestone would be added to it in recycling machine. After recycling and compaction, a thin HMA layer covers recycled layers. It protects the recycling layer from water ingress, traffic abrasion, restores the pavement structural capacity and smooth ride (Kim, Im, & Lee, 2010; Theyse, De Beer, & Rust, 1996).

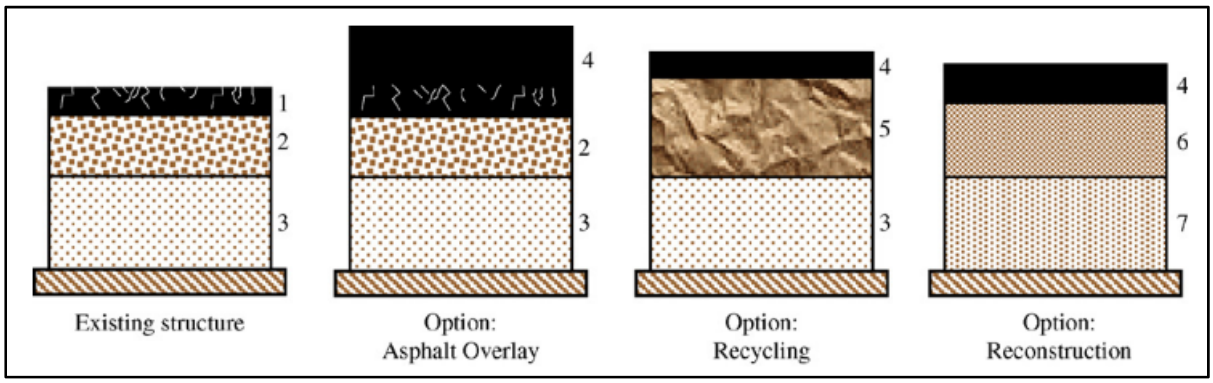

Figure 1.1 Diagram of the existing pavement and the different rehabilitation options (1) Existing distressed asphalt layer; (2) existing granular base; (3) existing granular subbase; (4) new asphalt layers; (5) recycled layer with foamed bitumen; (6) new granular bas
Taken from Thenoux et al (2007)

However, techniques and methods have been improving as well. To this end, recycling methods did develop and today there are many different ones. They all could be categorized in three major groups (Godenzoni, 2017):

- Hot in-place recycling (HIR);
- Cold in-place recycling (CIR);
- Full depth reclamation (FDR).

Different severity, type and level of damage in the pavement require different recycling and rehabilitation techniques. HIR is used for minimum extent of distress and is conventionally used for few upper centimetres of the pavement where there is no main structural deficiency

and it ranges between 20 to 50 mm. Although, depending on the equipment, it can reach as deep as 75 mm in depth (ARRA, 2001a; Godenzoni, 2017). Cold in-place recycling is normally involved with more severe, deeper and numerous distresses in the pavement which are non-load-related and have potential to extend further down into the pavement structure. However, a combination of CIR and overlay can be used to address load-related distresses (Stroup-Gardiner, 2011). FDR is completely the same as CIR and it takes place in-situ. The only difference is that some parts of the sub-layers of the pavement are also being recycled in FDR process and after mixing, they all are being used in as a new sublayer, treated and paved. FDR depth varies between 100 mm to 300 mm, depending on the thickness of the existing pavement structure (ARRA, 2001a; Xiao et al., 2018).

The reasons to justify the use of CIR techniques are adequately discussed. However, lack of sound standards for the mix design and application, beside deficient understanding about the behaviour and failure mechanism of cold mixes, especially foamed bitumen, prevent it from development in many cases. To this end, cold mixes are mainly utilized and practiced based on the experience (K. J. Jenkins, 2000).

Promisingly, there are a lot of researches which have been already conducted, and still ongoing, to determine the specification and behaviour of Cold Recycling (CR) mixtures to fill the gap of knowledge in this field (Apeagyei & Diefenderfer, 2012; Kim et al., 2010; Stimilli, Ferrotti, Graziani, & Canestrari, 2013).

1.2 Research problems

There are variable criteria which influence the final cold mixtures' properties such as RAP source, manufacturing process, stockpiling, emulsion or bitumen types, curing conditions, different volumetric properties and so forth. Although, in the last two decades the knowledge about the cold mixes is improved considerably, but there are still new grounds to break on CR, specifically in the optimization of the mix design and production process.

Curing is very important step in the production process which remarkably affects the final characteristics of the mix. However, the absence of comprehensive knowledge makes it to be an interesting topic for lots of research attempts to correlate laboratory and field curing conditions accurately. Laboratory-made specimens are supposed to represent the field. However, the test results for the laboratory-made specimens and the specimens cored out of the field are typically inconsistent. It is believed that some parts of this inconsistency come from curing conditions. Cored specimens are cured in a confined condition in which water doesn't leave out from the lateral side of the specimen, and this condition is not considered as a source of inconsistency.

Therefore, the problem is that we do not understand precisely the impact of the cure parameters. Moreover, considering a laboratory curing method which accurately imitates the field and confined curing condition in the field.

1.3 Research Objectives

This study attempts to adopt a new approach, to simulate the foamed asphalt mixtures field curing conditions in the laboratory. It's an endeavour to modify traditional laboratory curing conditions, to approximate the results to actual field conditions, which would probably provide us with a more realistic overview about it and would be helpful in matter of behavioural prediction.

Moreover, we considered two minor objectives attached to the major one:

- Investigating two different RAP source effects on the final strength of the material under the same proposed curing condition which leads to a more comprehensive understanding about it,
- Investigating different compaction methods and their corresponding impact on the strength of the materials.

1.4 Organization of the Thesis

This thesis is arranged in five chapters which start with the problem statement and objective of the thesis in the first one. At this very first chapter, it tries to start a discussion through roads, asphalt pavement and their rehabilitation. Afterward, it introduces the recycling concept and different techniques for rehabilitation, and it is followed by the problem statement and objective of the thesis. In the second chapter, a concise and complete literature review about cold recycling, cold mixtures and definitions about different kinds of them are presented. Cold mixture ingredients and the role they play in the mix are also included. Volumetric approach is reviewed, as well as different stages of the mix production. The third chapter is a full description of the material that are used in the laboratory, and different protocols and processes that have been exercised. In chapter 4, the results of the tests are fully presented, including test results and dimensions measurements. This is followed by the chapter five, which is titled "result analyzes and discussion". Conclusion is the final part of the thesis.

CHAPTER 2

LITERATURE REVIEW

This part of the thesis covers literature review. First, cold recycling methods are discussed and after that, different cold recycled mixes for the pavement structure are introduced and explained. It is followed by different ingredients of cold mixes. Volumetric properties of cold recycled mixtures (CRM), as an important part of the mix design, are also discussed subsequently, and chapter closes with information on compaction and curing of those materials.

2.1 Cold Recycling

Today, cold recycling uses new techniques, technologies and apparatus to mill the pavement, at different depths, and add different binders like bitumen emulsion or foamed bitumen. Lime or cement are also added to increase the mix characteristics. However, this is not a recent concept to use recycled material in road and pavement industry. (ARRA, 2001a) reported that using RAP and cold recycling in developed countries date back to 1900s. Documents firstly started to talk about recycling in beginning of the 1930s which was about usage of hot in-place recycling (HIR) (Godenzoni, 2017). Although, it took more 40 years, which was in the middle of 1970s, for recycling techniques and equipment to start developing (K. J. Jenkins, 2000). Two reasons pushed this development. First, companies were faced with scarcity of qualified virgin aggregate which meet the desired requirement (ARRA, 2001a). Second, the oil crisis in early of 1970,which meant an increase in all petrochemical productions including bitumen and oil price fluctuation (Stroup-Gardiner, 2011).

Lack of a sound guidelines for mix design and flaws in understanding about failure and behaviour mechanism hampered cold recycling development and limited its application into low traffic roads (Diefenderfer, Bowers, & Apeagyei, 2015; Muthen, 1998; Thomas & Kadrmas, 2003). Contrarily, HMA has been practiced by the scientists and different agencies for a long time and it made it to overshadow other methods like cold recycling.

The cold recycling application is through two different methods, in-plant and in-place. In in-plant method, recovered material is hauled to a depot plant and fed through a mixing unit. In in-place method there is no need for hauling, and everything is carried out through recycling machine (Wirtgen, 2012). In-place is cheaper and more environmentally friendly, since the hauling part is omitted, and reduces the risk because the pavement and recycled aggregates are not exposed to rain. However, in in-plant, there is more control on the variables and it leads to a required characteristics more precisely (Ramanujam & Jones, 2007; Xiao et al., 2018).

The material coming from milling the pavement or demolishing the old pavement is referred as reclaimed asphalt (RA) and can be re-added in the production of the new material (Godenzoni, 2017). RA is reused in HMA production structure. However, technical difficulties and environmental concerns limit this usage development (Grilli, Bocci, & Graziani, 2013). In order to achieve a recycled bituminous mixture which guarantees appropriate characteristics, RA needs to meet the requirements of the specifications and guidelines. There are multiple studies to highlight the importance of addressing two factors (Academy, 2009; ARRA, 2001a; Stroup-Gardiner, 2011):

- Quality control for adopting a production process and material,
- Characterization of RA according to European EN 13108-8standards (e.g. Amount and type of impurity existed in RA, age and type of the bitumen component, old and new bitumen content and type, original aggregate type and gradation).

CIR generally uses 100% of RAP generated in the process. Also, it is used to rehabilitate upper bound layers of the asphalt pavement, 50 mm to 100 mm, when the recycling agent is only asphalt emulsion (emulsified recycling agent). However, deeper depth rehabilitations between 125 to 150 mm are possible by CIR if additive like cement, lime or fly ash would be added to the mixture to improve the resistance to the moisture, and fast strength gain in the early stages after compaction. Moreover, required densification in CIR mixes needs more compaction energy in comparison to traditional HMA mixes and it’s due to the higher viscosity of the cold bitumen, lower compaction temperature and more inner friction angle between aggregate (Academy, 2009). After compaction, curing period can vary between several days and two

weeks and it is driven by environmental conditions, drainage and moisture characterization of the mix (Godenzoni, 2017). To be short, following advantageous come with CIR (Stroup-Gardiner, 2011):

- Construction benefits,
  - Minimizes traffic disruption,
  - Shortens lane closure time,
  - Maintains high clearance.
- Pavement condition improvements,
  - Improves friction,
  - Minimizes edge drop off concerns,
  - Reduces surface irregularities, distress type, severity, and extent,
  - Addresses some existing material problems such as moisture damage.
- Environmental benefits,
  - Conserves non-renewable resources,
  - Reduces emissions,
  - Reduces fuel consumption,
  - Reduces number of haul trucks,
  - Eliminates materials generated for disposal.
- Cost benefits.
  - Provides economical methods for pavement preservation and maintenance.

Recycled cold asphalt mixture is constituted of RA and often virgin aggregate, to modify the gradation, and bitumen in the form of emulsion or foam. Moreover, water presence in the mixture helps to heighten the workability by lubricating between the aggregates and enhances the distribution of the foam or bitumen emulsion (Tebaldi et al., 2012). More additives like cement and lime would be added to better the mixture properties, bitumen dispersion throughout the mixture and accelerate the curing (Tebaldi et al., 2014). Besides, cement and other active additives improve the curing speed and final characteristics of the mix (K. J. Jenkins, 2000).

2.2 Cold Recycled Mixtures (CRM)

Cold recycled mixtures treated with foamed asphalt or bitumen emulsion have a very similar composition than HMA. Water, which plays the same role as heat in HMA, is used to ease the laying down and compaction in Cold Mixtures (CM), controls rheological properties of the mix in its fresh state, leads the mix to its suitable volumetric properties and helps the better dispersion of the bitumen in the mix (Grilli, Graziani, & Bocci, 2012). Final characteristics (e.g. stiffness and strength) of CRM evolve in time and reach to their long-term state. This gradual evolution of characteristics is "*curing"* (Cardone, Grilli, Bocci, & Graziani, 2015; K. Jenkins, Ebels, Mathaniya, Moloto, & Mulusa, 2008). When RA is adopted to replace Virgin Aggregate (VA) within the CM body, the final product is Cold Recycled Mixture (CRM). The aggregate blend in CR can be a combination of different proportions of VA and RA according to the layers in which the mix would be used (subbase or foundation layer) (Godenzoni, 2017). CRM use bitumen as the binding agent. Since in low temperatures the viscosity of the bitumen is high, therefore, to have bitumen able to mix and compact in an ambient temperature, it is either placed between water particles through emulsification (i.e. via emulsifiers) or dispersed into the mix as foam, through steam (Tebaldi et al., 2014). A wide range of CRM has been produced using different combinations of RA, binding agents and additives (Thompson, Garcia, & Carpenter, 2009). Each mixture designed and optimized to meet the required characteristics according to the role it would play in the pavement structure (Kearney, 1997; Lewis & Collings, 1999; Maccarrone, Holleran, Leonard, & Hey, 1994). There are four different CRM families can be listed regarding the binder's content, physical and engineering characteristics (Grilli et al., 2012). The study used Asphalt Academy representation (Academy, 2002), and made comparison with traditional pavement material through this representation. Figure 2.1 is the diagram that illustrates this classification. So, four different categories of cold pavement mixtures can be identified according to this diagram:

- Cement-treated materials (CTMs),
- Bitumen stabilized materials (BSMs),
- Cement-bitumen-treated materials (CBTMs),
- Cold asphalt mixtures (CAMs).

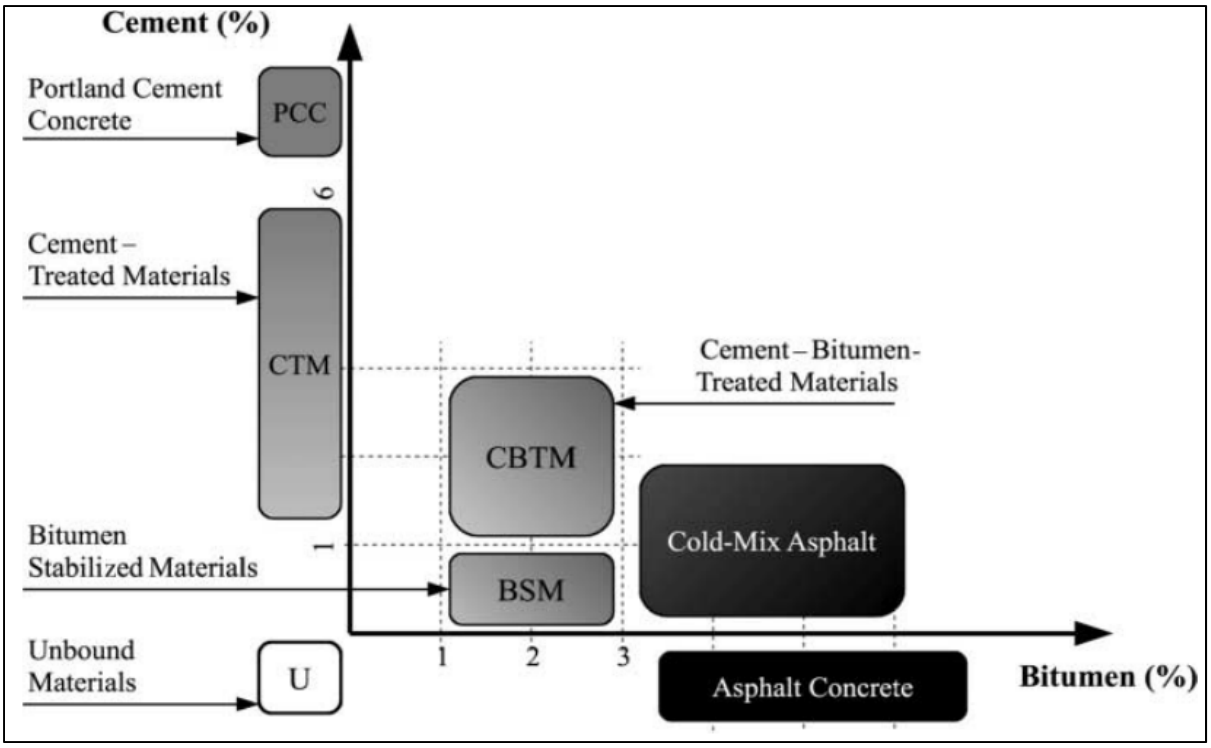

Figure 2.1 Conceptual compositions of the pavement mixture
Taken from Grilli et al (2012)

2.2.1 Cement-treated material (CTM)

There is just cement added as the binding agent in CTM material (Taha, Al-Harthy, Al-Shamsi, & Al-Zubeidi, 2002). CTM consist of selected aggregates and 2 to 5% of cement, by the dry aggregate weight (Grilli et al., 2013). Adding to this, defined percentage of water which is significant in the compaction and cement hydration (White & Gnanendran, 2005). Using cement in sublayers improves the stiffness in the sublayers significantly, and it helps better distribution of traffic loads and resists tensile strains under bituminous layer (Xuan, Houben, Molenaar, & Shui, 2012). Traditionally, CTMs used to use natural crushed aggregate in their structures. However, nowadays, it is increasingly associated with recycled material (Grilli et al., 2013). Although using VA in CTMs structure would result in mix with better mechanical characteristics, however, replacing VAs with RAs in the skeleton of the mix, and using a correct integration between these two to achieve proper grading, not only have its benefits, but

also it doesn't penalize the final mechanical characteristic of the mix considerably, compared to a fully VA composed mix (Grilli et al., 2013).

2.2.2 Bitumen stabilized material (BSM)

BSM uses bitumen, in shape of emulsion or foam, as the binding agent. Residual bitumen content in BSM is normally less than 3% of dry aggregate weight, and maximum 1% of cement as active filler is added to improve the strength of the mix without impairing the flexibility (Godenzoni, 2017). On one hand, B/C ratio needs to be bigger than one (Graziani, Godenzoni, Cardone, & Bocci, 2016), otherwise (Academy, 2009) considers it as cement treated material and there would be the fatigue failure concern (Wirtgen, 2012). On the other hand, using less than 3% cement in BSM structure improves its performance and acts as a catalyst in bitumen dispersion (Xiao et al., 2018).

Unlike HMA, there is no continuum bound between aggregate particles in BSM and coarser aggregates are not fully covered by bitumen, and bitumen tends to adhere only to finest particles which produce a bitumen-rich mortar between bigger aggregates and build localized non-continuous bounds. Therefore, BSM behaves like granular material using inter-particle friction with improved cohesion and should be regarded as granular material (Collings & Jenkins, 2011; Godenzoni, Graziani, & Perraton, 2017; Wirtgen, 2012).

2.2.3 Cement-bitumen treated material (CBTM)

CM with more cement in their structure, compared to BSM, up to 2.5% by the dry aggregate weight are referred as cement-bitumen treated material (CBTM) (M Bocci, Grilli, Cardone, & Graziani, 2011). They have a bigger B/C ratio, and better cohesion and stiffness compared to BSM (Grilli et al., 2012). The concept of CBTM is based on the CTM, and bitumen emulsion or foam is added to reduce the cracking vulnerability and permanent deformations and improve hardness of the layers in which recycled material is used. They are more like Asphalt Concrete (AC) in terms of temperature dependency and fatigue (Graziani et al., 2016; Grilli et al., 2012).

Bitumen is usually added in 2% to 3.5% by the dry aggregate weight. Mechanical behaviour of CBTM obeys the properties and the dosage of hydraulic and bituminous binders. Low dosage of cement and bitumen confers a granular-like behaviour to the mix. On the other hand, asphalt-like behaviour is recognizable for the mixes with more bitumen (bigger B/C ratio) (Academy, 2009; K. Jenkins, Long, & Ebels, 2007; K. J. Jenkins, 2000). Contrary, low B/C, or more cement presence in the mix design, results in stiffer behaviour which is vulnerable to shrinkage cracking and brittle behaviour which is closer to that of CTM (M Bocci et al., 2011). Adding hydraulic binder to the CBTM improves the early life characteristics of the mix and it is very useful in the field. Moreover, bitumen covers the fine particles in the mix and enhance their moisture resistance (M Bocci et al., 2011). Strength, stiffness, permanent deformation, cohesion, temperature dependency, durability, moisture resistance and fatigue are controlled by bitumen and cement ratio. To this end, an optimum binders ratio is very important aspect in the mix design (Godenzoni, 2017). Although CBTM and HMA have fundamental differences, however, similar approaches can be used to analyze their mechanical behaviour (Montepara & Giuliani, 2002; Santagata & Chiappinelli, 2003).

2.2.4 Cold asphalt mixtures (CAM)

High bitumen dosages, compared to BSM and CBTM, and a limited cement amount are used as binders in cold asphalt mixtures (CAM) (Graziani et al., 2016). Bitumen content ranges between 3 to 6% of dry aggregate weight and less than 2% of cement content as an active filler. It makes the B/C ratio range between 1.5% to 3%, which denotes resembling characteristics of AC, like thermal dependency and flexibility (Godenzoni, 2017). Despite the differences between HMA and CAMs, similar approaches can be adopted to analyze both (Montepara & Giuliani, 2002; Santagata & Chiappinelli, 2003). Table 2.1 is a concise explanation of the constituent content of the different types of CM.

Table 2.1 Ingredients of the different types of the cold mixtures
Adapted from Grilli et al (2012)

CM type	Added cement %	Added bitumen %	B/C	Specifications
CTM	2 to 5	-	-	Brittle behaviour, used in base course
BSM	≤ 1	≤ 3	≥ 1	Granular behaviour
CBTM	≤ 2.5	≥ 2.5	≥ 1	Close mechanical behaviour to AC
CMA	≤ 2	3% to 6%	≥ 1.5 to 3	Close mechanical behaviour to AC

2.3 Cold recycled mixtures' ingredients

CM could be described as a multi-phase material, since they are constituted of different material from different phases as follows:

- Solid; virgin or reclaimed aggregates, and bitumen (once it is cured in the mixture),
- Fluid; water and bitumen in shape of foam or emulsion,
- Air voids.

Since this study focusses on the foam bitumen, CM refers to CM with foam bitumen thereupon.

2.3.1 Solid phase

Solid phase constitutes the skeleton of the mixture and is the main load-bearing structural part of the mix. It includes aggregates, filler, cured bitumen and hydrated cement.

2.3.1.1 Material Type

There is a wide range of aggregates, including both virgin and reclaimed, that can be used in the foam asphalt structure. However, these different kinds of the aggregates may require

gradation adjustment to fulfil the expected properties (i.e. it needs fine or coarse aggregates to be added to achieve the required gradation curve) (Muthen, 1998). On top of that, since the expected requirements of the mixtures with or without RA material are the same, in practice, the level of the homogeneity of the material defines the allowable amount of RA to use. Inhomogeneity in the RAP would multiply the inhomogeneity in the mix. To this end, the more scattered the properties are in the RAP, the lower amount of the RAP would be allowable to be used in the mix (Godenzoni, 2017; Tebaldi et al., 2014).

Maximum density (Fuller & Thompson, 1907) gives the densest gradation, and at the same time serves the adequate space to bitumen binders that secures workable mix and required optimum characteristics. To this end, it is assumed as an evaluative benchmark (Mallick & El-Korchi, 2013). However, (Bailey, 2002) introduced a grading method in which aggregates are sorted into four classes. Stone, interceptor, coarse sand and fine sand. Each of them plays their specific role in the mix. (Olard & Perraton, 2010) used this method and optimized the grading based on the filling the air voids between the coarse aggregates with finer aggregates to improve the mix characteristics. Moreover, it is worth saying that CRM can surprisingly act paradoxically, in matter of workability and compactability. In another word, CRM with workable characteristic, which is expected to get compacted easily, requires more compaction energy (Raschia et al., 2019). These two contradictory factors are hard to optimize simultaneously. Furthermore, excessive water content washes the fine particles and bitumen with it during the compaction (Grilli, Graziani, Bocci, & Bocci, 2016).

However, Mobil Oil established a guideline to choose the proper gradation to use for foam asphalt mixtures (Akeroyd, 1989). Beside grading suitability, there are also some designing points about the binder content for ideal graded material.

Other material qualities are necessary for an accurate mix design and laboratory testing, which are used to refine the mix design and take the volumetric properties into account. To this end (Bowering & Martin, 1976) utilized grading features to define the optimum foam bitumen

content for the foam mixes. Following them, (Ruckel, Acott, & Bowering, 1983) introduced Table 2.2 to rank variable material in order to use in foam mixes.

Table 2.2 Suitability of the material for foam treatment
Taken from Ruckel et al (1983)

Soil Type	**Suitability for foamed mix**	**Optimum bitumen content (% m/m)**	**Comments**
W.G gravel, little or no fines	Good	2-2.5	Permeable (improve with crushed fraction)
W.G gravel + some clayey silt	Good	2-4	Permeable (improve with crushed fraction)
W.G gravel + sandy silt	Good	2-4	Permeable (improve with crushed fraction)
P.G gravel + sandy clay	Good	2.5-3	Low permeable (improve with crushed fraction)
Clayey gravel	Poor	4-6	Improved with lime
W.G sand	Fair	4-5	Needs filler
W.G silty sand	Good	2.5-4	
P.G silty sand	Poor	3-4.5	Use lower pen bitumen, add filler
P.G sand	Fair	2.5-5	Needs filler
Silty sand	Good	2.5-4.5	
Slightly clayey, Silty sand	Good	4	
Clayey sand	Poor	4-6	Needs small % lime
	Good	3-4	After lime modification
Note! W.G = well Graded, P.G = Poorly Graded			

In the pavement structure aggregate size ranges between 0.075 mm and 70 mm and RAP aggregate size can be between 19 and 75 mm (K. W. Lee, Brayton, & Huston, 2002). Although, chemical properties of the aggregates need to be considered in the design process as well

(Mallick & El-Korchi, 2013). RA binder type and content are less important since RA aggregates are supposed to act as *black rock* (Soleymani, McDaniel, & Abdelrahman, 2000). Most of CR projects happen without adding any new aggregate. The decision to add new aggregate should not be based just on the RAP gradation test, however, adding new aggregate where excessive binder content is present, may be justifiable and beneficial. Moreover, final blend of aggregates needs to meets the requirements (ARRA, 2001b).

When foam asphalt touches recovered material, it fragments into millions of globules which prefer to adhere to the finer particles (i.e. fine sand and smaller in the mix), and compose a filler mortar which binds the coarser aggregates (K. J. Jenkins, 2000).

Filler content is the most determining factor in choosing optimum binder content in foam mixes since it is too critical to achieve the highest strength. It preferably should be more than 5%. What filler term refers to, is the finest particles of the aggregates with the diameter smaller than 0.075 mm (Akeroyd, 1989; Bowering & Martin, 1976; K. Jenkins, Van de Ven, & De Groot, 1999; Ruckel et al., 1983). To this end, (Ruckel et al., 1983) proposed Tables 2.3 in which foamed bitumen content is obtained as a function of coarse and fine aggregates content. The finer the aggregates are, the more foamed asphalt content is recommended.

The bigger ratio of the bitumen to fine aggregate decreases the final strength of the mix. A big part of the CMs strength comes from friction between coarse aggregate and this is one of the main differences between foam mix and HMA. It illuminates two facts; first, foam mixes are more resistant to temperature changes, and second, grade of the bitumen used for foaming is not critical (Muthen, 1998).

Table 2.3 Foam bitumen content
Taken from Ruckel et al (1983)

Passing 4.75mm Sieve (%)	Passing 0.075mm Sieve (%)	Foamed bitumen content (% m/m dry aggregate)
<50	3.0 - 5.0	3.0
	5.0 - 7.5	3.5
	7.5 – 10.0	4.0
	>10.0	4.5
>50	3.0 - 5.0	3.5
	5.0 - 7.5	4.0
	7.5 – 10.0	4.5
	>10.0	5.0

Beside VAs (Virgin Aggregates), other additives (e.g. cement, lime or fly ash) would be added, usually from 1% to 2.5% by the aggregate weight, to speed up the curing process and improve the mix characteristics (M Bocci et al., 2011; Grilli et al., 2012). The main purpose of adding cement into the mixture is to increase the compressive and tensile strength or reducing plasticity (Du, 2014; Godenzoni, 2017). Cementitious stabilization agent is so determining in the final strength of the mix. On the other hand, it plays catalyst role in dispersion of the bitumen (Muthen, 1998; Wirtgen, 2012; Xiao et al., 2018). As cementitious ingredient increases, beside strength, brittle behaviour also increases and flexibility decreases. So, the compacted layer can distribute more load under the wheels. However, excessive cementitious content increases the risk of shrinkage cracking (Grilli et al., 2013; Little & Petersen, 2005; Xiao et al., 2018).

The moduli of the foam treated materials are lower than HMA (Loizos, 2006); although, it is more than cement or lime treated material (Sakr & Manke, 1985). Moisture sensitivity is also improved through cement treatment (Bissada, 1987; Saleh, 2004). Particularly, more than 2% cement in the mix, categorizes the mix as CBTM (Maurizio Bocci, Grilli, Cardone, & Ferrotti, 2014; M Bocci et al., 2011). At low cement contents the product of the cement hydration,

disperses inside the bituminous mortar and beside rising the viscosity, improves the resistance against the permanent deformation. Higher cement additive forms stiffer matrix which connects coarser aggregates in the mix structure (García, Lura, Partl, & Jerjen, 2013).

2.3.2 Fluid phase

As it was mentioned, fluid part refers to bitumen (in the shape of foam or emulsion before breaking or setting down), and water.

2.3.2.1 Bitumen

Bitumen is the main binder in the CRM structure. Since the viscosity of the bitumen is high at the low temperatures, which makes the utilization harder, it is added in the shape of emulsion or foam (Raschia et al., 2019). Using bitumen in CM improves cohesion (shear strength), fatigue resistance, moisture sensitivity and prevents ravelling problem (Muthen, 1998). In emulsions, intense shearing stress is used to break the bitumen into small droplets, and emulsifier wraps the bitumen droplets and imposes an electric charge which helps the bitumen particles to repel each other and prevent them from gathering, hardening and decreases the viscosity of the bitumen in a large extent (Tebaldi et al., 2014). Foam bitumen is produced by injecting cold water molecules and air into the hot bitumen, in an expansion chamber. This is a physical process rather than a chemical (Csanyi, 1959; Ruckel et al., 1983), Figure 2.2.

The foam state at an ambient temperature is a very short period, less than a minute. It allows the bitumen content to get dispersed evenly throughout the aggregates at the ambient temperature and moisture content. After this short period, it resumes its original properties (He & Wong, 2007; Iwański & Chomicz-Kowalska, 2013; K. Jenkins et al., 1999; Muthen, 1998). Regardless to grade and origin of a bitumen, any type of bitumen could be foamed with a proper combination of nozzle type, water, air and bitumen injection pressure (Castedo-Franco, Beaudoin, Wood, & Altschaeffl, 1984). However, type and grade of the bitumen, and additives

(e.g. anti foaming agent) affect the foaming characteristics (Saleh, 2007). Foamed bitumen should be applied into the aggregate while it is still in foam state (Muthen, 1998).

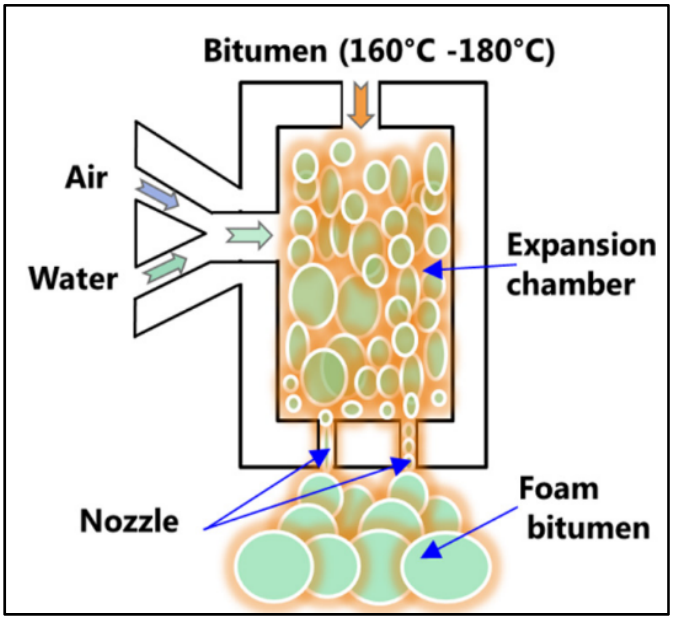

Figure 2.2 Production of the foam bitumen in expansion chamber
Taken from Hailesilassie et al (2015)

Due to the selective nature of the foam bitumen to stick to the fine aggregates (i.e. fine sand and smaller), foam bitumen constructs a non-continuous structure while very small amount of foam bitumen adheres to coarse aggregates. In fact, small foam bubbles have only enough energy to warm up smaller particles which allows adhesion, Figure 2.3 (K. J. Jenkins, 2000).

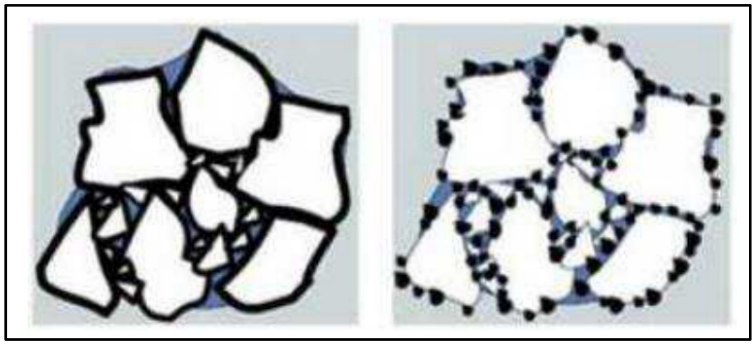

Figure 2.3 Continuous and non-continuous bonds
Taken from Godenzoni (2017)

Foam bitumen is evaluated via two properties, Expansion Ratio (ER) and half-life ($\tau_{1/2}$). ER is the ratio of the maximum volume of the foamed bitumen to the bitumen volume which is used for foaming. Half-life is the time that the maximum volume of the produced foamed bitumen would cut in half as it contracts (Wirtgen, 2012). Optimum foaming water content is the one in which the ER and half-life would be maximum simultaneously (Iwański & Chomicz-Kowalska, 2013). There are no standard limitations of these two values, however, there are some recommendations in the literature (Academy, 2009; S. Acott & Myburgh, 1983; Ruckel et al., 1983; Wirtgen, 2012).

Lots of factors affect ER and half-life. The important ones are (Brennen, Tia, Altschaefl, & Wood, 1983):

- The amount of foam produced,
- Foaming water addition,
- Foaming temperature of the bitumen.

More foaming water and foaming temperature increase ER and decreases half-life (Muthen, 1998), although, they affect in different magnitudes (H. D. Lee & Kim, 2003). (Bowering & Martin, 1976) showed that the bigger expansion ratio (15:1) levels up the compressive strength to a great extent. Moreover, the bigger ER value improves the aggregate coating, and consequently, enhances the properties (Maccarrone et al., 1994).

Moisture content in the aggregate, before adding foamed bitumen, is a crucial design factor in foam mixes (D. Lee, 1981) since:

- It carries the binder during the mixing process which results in better dispersion of bitumen in the mix,
- Easing factor for compaction, workability and required density accomplishment,
- Promoting the shelf-life.

In fact, the water inclusion in the foam mixes is what separates these mixtures of HMA (K. J. Jenkins, 2000). Water makes the finer particles suspend and it provides a network for the foam bitumen to travel and disperse better through all parts of the mix (Csanyi, 1959). Low moisture content obstructs the compaction, workability and dispersion of the foam in the mix (Bowering

& Martin, 1976). On the other hand, too much moisture prolongs curing, lowers the density and strength (Brennen et al., 1983).

2.4 Mix design Consideration

Material mainly comes from the natural resources with different variance, availability and suitability according to the application area; and, there are different expected behaviours of the composite material in each case. To this end, mix design is necessary. However, limited applications and secretive approaches adopted by different operators left design and process guidelines and standards unimproved (K. J. Jenkins, 2000).

The final purpose of the mix design in the laboratory is to reach an optimum composition contents of the mix which fulfils the required characteristics. Moreover, the proportions and properties of the mix are needed to be considered economically to yield the optimum mixture specifications. Mix design considerations according to Asphalt Institute are (Sunarjono & Hidayati, 2018):

- The bitumen content which supposed to serve durability and fatigue cracking resistance of mixtures;
- The stability and stiffness modulus of mixture which withstand deformation caused by traffic loads;
- The void percentage to maintain the mixture from flushing, bleeding, or loss of stability during secondary compaction under traffic loading;
- Mix workability during mixing, laying and compaction process.

Volumetric and compaction are important in the mix design optimization. Moreover, engineering properties, longevity and performance of the material during its service life are essential. On the other hand, optimization is strongly linked to the economic consideration as one of its main objectives (K. J. Jenkins, 2000). From all the aspects to consider in an optimized mix design, performance is the hardest one to fulfil because of the following factors:

- Variability of methods to evaluate the performance of material, viz durability, fatigue.

- Problems associated with recognition of the material failure mode and the mechanism which results in failure,
- Variable mix specifications and material natural properties which have an impact on the mix final performance (e.g. gradation and aggregate hardness),
- External factors diversity like loading magnitude and speed,
- Difficulties in the long-term behaviour simulation due to lack of unique method for every specific property of material,
- Long term behavioural investigations time and budget consuming.

Foamed asphalt mix design purpose is to proportionate different ingredients of the mix to fulfil three goals (Muthen, 1998):

- Choosing the optimum quantity of different contents of the mix,
- Meet in-field service requirements,
- Make sure the engineering properties will remain as it is designed for, in different temperature, moisture and loading conditions.

2.5 Volumetric properties

Previously, volumetric analyzes for foam asphalt mixtures were either focused on BSM mixes with foam bitumen (particularly, the mineral filler content) (K. J. Jenkins, 2000), or traditional mix design method for AC (Cominsky, Huber, Kennedy, & Anderson, 1994). Moreover, binder properties were also found less variable than volumetric properties. Volumetric properties could be affected by compaction temperature and aggregate gradation. However, final concern is that whether RAP incorporation in CRM structure affects the mechanical and volumetric properties or not (Soleymani et al., 2000).

As the binder content increases, density increases and voids content decrease, hence, it is expectable that density and mix volumetric could be utilized to define the optimum binder content of the mix (Muthen, 1998).

There are two main factors which discriminate CRM and HMA in matter of composition and behaviour. First, water presence in the CRM which plays the same role of the heat in HMA. They both facilitate the compaction process. In fact, moisture content controls rheological properties of CRM in its fresh state. In other words, it is the main element which leads the mix to the desired volumetric properties (Grilli et al., 2012). Second, curing phenomena in CRM which makes their structure and mechanical behaviour to evolve through the time until their long-term state (Gómez-Meijide & Pérez, 2014; K. Jenkins et al., 2008).

Volumetric properties have a deep impact on the mechanical characteristics, engineering properties and durability of the CBTM (Cominsky et al., 1994). On the other hand, different projects come with the specific requirements in terms of function and different features of the final product, hence, it needs a meticulous consideration in both mix design and quality control phases (Grilli et al., 2016).

Moreover, the heterogeneity of the components of the cold mixture and the interaction between them make cold mixtures different than traditional HMA (M Bocci et al., 2011; Stimilli et al., 2013). Their mechanical behaviour and characteristics are combination which come from HMA and unbounded granular material (Khosravifar, Schwartz, & Goulias, 2015).

Traditionally, volumetric properties of CTM and granular material are based on dry bulk density ρ_d (Grilli et al., 2016). However, since this approach does not include bitumen (emulsion or foam) in it, and bulk density is mostly based on the each present particle density in the mix, it is inappropriate to use this approach for mixes in which RA is used, and reformulating seems necessary since the dry density of VAs and RAs are definitely different (Kuna, Airey, & Thom, 2014).

There are tons of combinations of virgin aggregates, RA, cement, water and bitumen that each of which produces a different CBTM with different characteristics. Therefore, accurate definition of CBTM volumetric properties is required to analyze the mix in its fresh state and cured state (Grilli et al., 2012).

From all the water in the mix design, a portion fills the permeable parts of the aggregate and the rest of the water, included post-added water and water in the emulsion (or foam), helps compaction as a free liquid phase. To be noted, it's necessary to add any additional water before cement or bitumen to homogenize and saturate the aggregate void parts (Grilli et al., 2012).

To start formulating volumetric for CBTM, three definitions are introduced in equations 2.1 to 2.3. Free water content (W_F), residual bitumen content (B_R) and free liquid content (L_F). All the terms are depicted in Figure 2.4.

$$W_F = \frac{M_{W,F}}{M_{B,R} + M_S} . 100 \quad (2.1)$$

$$B_R = \frac{M_{B,R}}{M_{B,R} + M_S} . 100 \quad (2.2)$$

$$L_F = \frac{M_{B,R} + M_{W,F}}{M_{B,R} + M_S} . 100 \quad (2.3)$$

Where;

$M_{W,F}$: Mass of free water (water not absorbed by aggregate, either as a part of emulsion or added water).

$M_{B,R}$: Mass of residual bitumen in the emulsion

M_S: Total mass of solid phase (aggregate, cement and filler)

Solid phase of mixes consists of RA, which is a mix of RA_g (Recycled aggregate from cement bounded or unbounded layers), VA (virgin aggregate) from different natures, fillers and cement. Permeable parts of the aggregate should be included in the calculations, and relevant particle density for each particle should be considered. For example, ρ_a(Apparent particle density) should be replaced by ρ_{ss}(saturated surface dry density) which makes it easier to work

with and considers absorbed water weight. Both of ρ_a and ρ_{ss} calculations comply with EN 1097-6.

ρ_m(Maximum density) is the first definition to describe volumetric properties of CBTM mixtures, which is the mass per unit volume of the mix at zero voids, (Equation 2.4).

$$\rho_m = \frac{\sum_i M_i}{\sum_i V_i} \tag{2.4}$$

M_i is the mass of constituent material, and V_i is their corresponding volume (Figure 2.4). Since due to evolutive behaviour of CBTM mixtures, the mass of the water changes, so it should not be considered in ρ_m calculations. ρ_m could be calculated by following formula which complies with EN 12697-5 9, (Equation 2.5):

$$\rho_m = \frac{100}{\sum_i \frac{P_i}{\rho_i}} = \frac{P_{VA_g} + P_{RA} + P_{RA_g} + P_C + P_{B,R}}{\frac{P_{VA_g}}{\rho_{a,VA_g}} + \frac{P_{RA}}{\rho_{a,RA}} + \frac{P_{RA_g}}{\rho_{a,RA_g}} + \frac{P_C}{\rho_C} + \frac{P_{B,R}}{\rho_B}} \tag{2.5}$$

Where;

P_i is each constituent material content as a percent of the total mixture mass and ρ_i is its density. ρ_a (Apparent density) is for aggregates. The bitumen in the RA is included in $\rho_{a,RA}$ and residual bitumen in emulsion is separately considered. Maximum density of CBTM mixtures could be considered as a constant quantity during service life like other bituminous mixtures. However, due to uncertainty associated with different constituents of these mixtures, the variability is larger in comparison to other bituminous mixtures. Bulk density in CBTM mixture is defined as follows, (Equation 2.6):

$$\rho_b = \frac{\sum_i M_i}{V_{\mathrm{T}}} \tag{2.6}$$

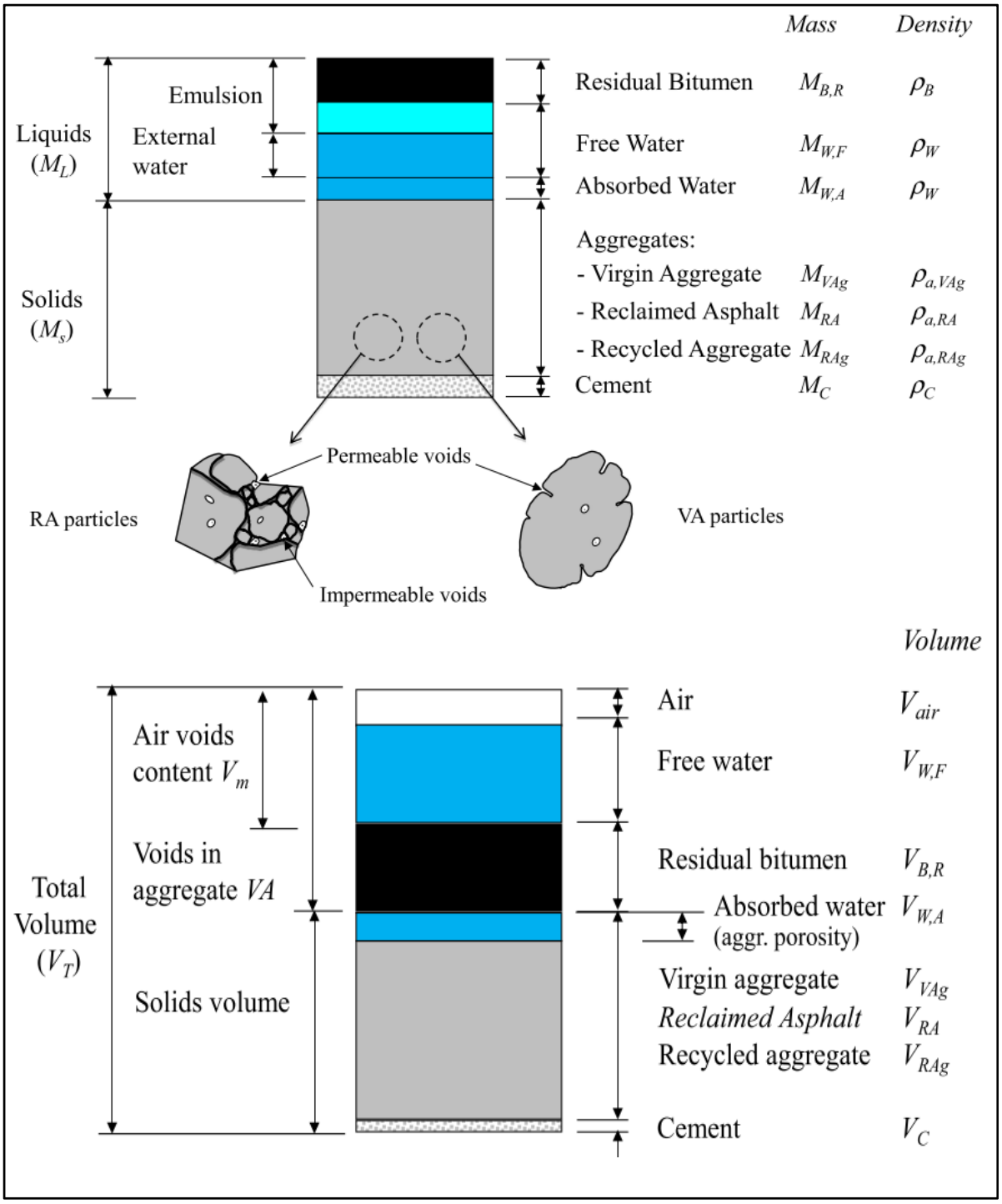

Figure 2.4 Constituents materials and volumetric characteristics of the CBTMs:
(a) constituents by mass and (b) constituents by volume
Taken from Grilli et al (2012)

M_i represents the mass of each component material (including water), and V_T is the total volume of the mix, including air voids, (Figure 2.4). Bulk density can also be used to evaluate the degree of saturation of the mixture, particularly in the fresh phase of the mix in which it has not been compacted yet and bitumen alongside water works as a single liquid phase. After the breakdown of the emulsion, bitumen starts to contribute in the structure of the mix, while water reacts with the cement, and after that, the quantity of water decreases due to curing in service life. Unbound material, CTM and BSM use dry density in their volumetric relationships, which is dry mass weight divided by their volume. After curing, the bitumen segment of the emulsion sits down in the structure of the mix and burdens some parts of the mechanical characteristics in the mix. So, for calculating the dry density, the mass of residual bitumen would be added into solid phase weight (Equation 2.7):

$$\rho_d = \frac{M_{\mathrm{S}} + M_{\mathrm{B,R}}}{V_{\mathrm{T}}} = \frac{M_{\mathrm{VA_g}} + M_{\mathrm{RA}} + M_{\mathrm{RA_g}} + M_{\mathrm{C}} + M_{\mathrm{B,R}}}{V_{\mathrm{T}}} \tag{2.7}$$

The typical approach used to evaluate the volumetric properties and compactability of granular mixtures and cement-treated mixtures is based on the dry bulk density ρ_d.

Using dry density (ρ_d) to do the mix design in CBTM mixtures and their compactability evaluation could be inappropriate since the density of virgin aggregate and recycled aggregate and their proportions in different mixes could be highly variable. Current approaches use volumetric evaluations to evaluate CBTM mixtures in practice. To this end, voids in the aggregate (VA) are proposed in equation 2.8.

$$\mathrm{VA} = \frac{V_{\mathrm{V}} + V_{\mathrm{W,F}} + V_{\mathrm{B,R}}}{V_{\mathrm{T}}} . 100 \tag{2.8}$$

V_v is the air voids volume, $V_{W,F}$ is the free water volume (not absorbed by the aggregates), $V_{B,R}$ is the residual bitumen volume and V_T is the total mixture volume. Voids in the aggregates provide a way for the volumetric evaluation of the solid. Using equations 2, 3 and 8, equation 2.9 can be deduced.

$$\mathrm{VA} = V_{\mathrm{m}} + \frac{\rho_{\mathrm{d}}}{\rho_{\mathrm{B}}} . B_{\mathrm{R}} \tag{2.9}$$

V_{m} in equation 9 is the content of free water and air voids in the total mixture volume (non-structural part volume).

$$V_{\mathrm{m}} = \frac{V_{\mathrm{v}} + V_{\mathrm{W,F}}}{V_{\mathrm{T}}} . 100 \tag{2.10}$$

Water fills part of this volume (unsaturated condition) or all of it (saturated condition). However, water should be excluded of the calculations for characterizing CBTM mixture or compaction effectiveness. Therefore, V_{m} is calculated by using dry density (equation 2.7) instead of bulk density.

$$V_{\mathrm{m}} = 100 . \frac{\rho_{\mathrm{m}} - \rho_{\mathrm{d}}}{\rho_{\mathrm{m}}} \tag{2.11}$$

The part of VA filled with effective liquids (water and bitumen), before emulsion breaking, is another important volumetric base for CBTM. In this matter, volume of the voids filled with liquids is important:

$$\mathrm{VFL} = \frac{V_{\mathrm{W,F}} + V_{\mathrm{B,R}}}{V_{\mathrm{V}} + V_{\mathrm{W,F}} + V_{\mathrm{B,R}}} . 100 \tag{2.12}$$

Using equation (2.1), (2.2) and (2.7), equation (2.12) could be replaced by equation (2.13):

$$\mathrm{VFL} = \left(W_{\mathrm{F}} \frac{\rho_{\mathrm{d}}}{\rho_{\mathrm{W}}} + B_{\mathrm{R}} \frac{\rho_{\mathrm{d}}}{\rho_{\mathrm{B}}} \right) . \frac{100}{\mathrm{VA}} \tag{2.13}$$

And if $\rho_{\mathrm{W}} = \rho_{\mathrm{B}}$

$$\mathrm{VFL} = \left(L_F \frac{\rho_{\mathrm{d}}}{\rho_{\mathrm{w}}} \right) . \frac{100}{\mathrm{VA}} \tag{2.14}$$

Equation 2.13 and 2.14 are like saturation degree. Be noted, permeable particles void is considered saturated.

2.6 Compaction

Compactability in foamed asphalt mixes differs from HMA due to two important factors (K. J. Jenkins, 2000):

- Binder distribution within foamed asphalt mixes and HMA,
- Water presence in foamed asphalt mixes which differentiates its nature with HMA.

Various laboratory compaction methods were previously applied by different studies, including Marshall compaction (Brennen et al., 1983; Kim & Lee, 2006; Muthen, 1998; Xu, Hao, Ma, & Liu, 2012), vibratory compaction (Bowering & Martin, 1976; K. J. Jenkins, 2000; Shackel, Makiuchi, & Derbyshire, 1974) and gyratory compaction (Brennen et al., 1983; K. Jenkins, Robroch, Henderson, Wilkinson, & Molenaar, 2004; Maccarrone et al., 1994). However, the Ideal compaction method is the one which represents the field more precisely. To this end, modified proctor density is used as a reference density worldwide since it reveals the closest density to the field compaction (Kuna, Airey, & Thom, 2017).

Unlike gyratory, there are many well-established guidelines and standards for Marshall, vibratory and proctor compaction. However, lots of studies are conducted to investigate gyratory details, including pressure and angle of gyration, Table 2.4. These studies have evaluated the feasibility of using laboratory gyratory compaction on the foamed bitumen mixtures (FBM) (Kuna et al., 2017). To introduce a new compaction procedure with gyratory machine, 600 kPa and 1. 25° as gyration angle was proposed as the result of these studies. The work of Brosseaud (Brosseaud, Gramsammer, Kerzreho, Goacolou, & Le Bourlot, 1997) is probably the most promising one, which used gyratory compaction technique to simulate the field compaction, and relates laboratory and field compactions for several foamed mixes.

Table 2.4 Gyratory compaction efforts on foam bitumen mixes
Taken from Kuna et al (2014)

Study	Number of Gyrations (N)	Compaction Pressure (MPa)	Compaction angle (Degrees)	Reference Density
Brennan (1983)	20	1.38	NA	2.25 kg/m^3
Maccarone et al. (1994)	85	0.24	2	Field density
Jenkis et al. (2004)	150	0.6	1.25	Modified Proctor density
Kim and Lee (2006)	30	0.6	1.25	Marshall Density (75 blows)
Saleh (2006)	80	0.24	2	Australian guideline for HMA
Note: NA = not available				

2.7 Curing

Curing, whether for emulsion or foam mixes, is the process in which water leaves out of the specimen body, by evaporation and through time. This moisture loss results in stiffness and strength growth (tensile and compressive) (K. J. Jenkins, 2000).

Bowering (Bowering, 1970) states that full-strength development of CRM would not happen unless a large percentage of moisture of the compacted mix would be discharged. The curing rate, final cured state and strength characteristics development of the CRM depend on ambient conditions like temperature and relative humidity (Cardone et al., 2015; Godenzoni, Cardone, Graziani, & Bocci, 2016; Graziani, Iafelice, Raschia, Perraton, & Carter, 2018).

In CRM with foam, the change in the moisture content happens only through curing. Curing plays a vital role since strength would not be fully developed in the mix structure before curing takes place thoroughly. Only after that, the mix can undergo the traffic load. Curing in foam

asphalt mixture is due to migration of water due to compaction, moisture repulsion by the bitumen and moisture loss through evaporation (Academy, 2009).

Cement hydration also contributes to curing, since it confers strength to the mix (Maurizio Bocci et al., 2014). On the other hand, in early stages of the curing cement fosters foam bitumen coalescence via reducing free water content (García et al., 2013; Terrell & Wang, 1971; Wang, Liu, & Hu, 2013). However, bitumen presence, foam or emulsion, decreases cement hydration rate and it hampers the cementitious structure to establish thoroughly (K. J. Jenkins, 2000; Muthen, 1998).On the other hand, curing phenomenon in CRM is tightly linked to the moisture evaporation while cement hydration requires water presence and does not entail moisture loss (Cardone et al., 2015).

Understanding the foam asphalt curing is necessary, since (Fu, Jones, Harvey, & Halles, 2009):

- It helps to develop standard curing procedures in the lab, which represent the field accurately,
- It helps to improve the curing process and expedite early strength development, and meanwhile, optimizes long-term performance.

All proposed curing procedures in the lab targets the same goals. First, they try to mimic the field curing condition accurately. Second, since it takes months for curing to complete in the field, realistic simulation of the field in the lab is infeasible, so accelerating the attainment of final characteristic is the second target (K. Jenkins et al., 2008). Repeatability and reproducibility are considered as the third feature to establish a curing method. In other words, one should be able to repeat the test for the same material, in the same laboratory, with the same procedure and operators (repeatability), or in different laboratories with different operators (reproducibility) (Fu et al., 2009). However, in each experiment, there are always some factors beyond control which need more attention to have the minimum bias impact on the results (Fu et al., 2009).

To understand the curing (Fu et al., 2009), Figure 2.5, describe four different phases of curing chronologically.

1. Foam bitumen is added into the moist blended aggregate and it binds fine aggregate to form an asphalt mastic, Figure 2.5.a;
2. Aggregates in loose mix is coated with a water membrane and after compaction, they touch asphalt mastic tightly and some bonds may develop, Figure 2.5.b;
3. Because of the water presence, asphalt mastic and aggregate bond stay weak and would not develop to its full until the water evaporates, Figure 2.5.c and d;
4. Water in larger voids evaporates first as the curing progresses. Water evaporation out of finer voids and especially finer voids in the mastic phase and aggregates interface is harder due to their low thermodynamic potential. However, after bonds between asphalt mastic and aggregates develops and is fully formed, the mix exposure to the water doesn't affect these bonds considerably, Figure 2.5.e.

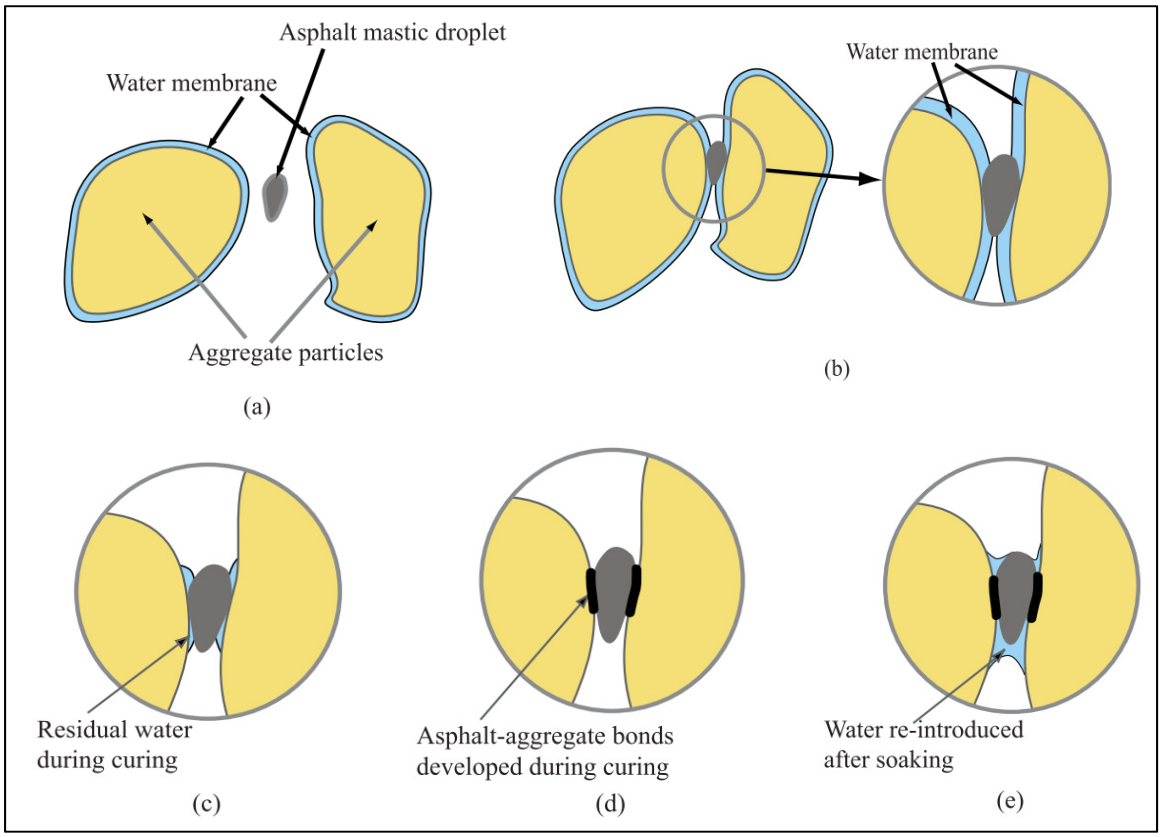

Figure 2.5 Conceptual depiction of the foam asphalt curing
a) moist aggregate and asphalt droplet b) Post compaction state c) water evaporation d) bond development, and e) reintroducing of water into the foam mix
Taken from Fu et al (2009)

2.7.1 Curing protocols

Air temperature, humidity and underlying material conditions are affective factors in curing. Since these factors are open to change along the road and over the time of construction, it is impractical to replicate the field conditions in the lab exactly (Fu et al., 2009).

Required curing time span is a function of humidity, temperature and traffic. In other word, at least a full cycle of four seasons is required to complete the curing. Although, it is changeable according to the ambient conditions and traffic (Serfass, Poirier, Henrat, & Carbonneau, 2004). However, in the laboratory, typical curing periods range from several days to 2 weeks, depending on the above-mentioned factors, recycling additive and any modifiers used (ARRA, 2001a; Stroup-Gardiner, 2011; Thompson et al., 2009). Every accelerating curing condition requirements are (Serfass et al., 2004):

- To be short time wisely,
- It should result in a same material as it would in the site,
- It should avoid bitumen binder aging as much as possible,
- Laboratory equipment should not be sophisticated.

2.7.2 Temperature

Temperature is previously mentioned as one of the main factors in the curing. Bowering (Bowering, 1970) proposed curing at 60 °C for 72 hours and it was followed in many other studies (S. M. Acott, 1980; Bowering & Martin, 1976; Lancaster, McArthur, & Warwick, 1994; Maccarrone et al., 1994; Muthen, 1998). Ruckel (Ruckel et al., 1983) offered 40 °C and 24h to simulate the first 7 to 14 days of construction, and 40 °C for 72 hours to simulate the long term curing after construction (>30 days). Ambient temperature was also introduced for curing foamed bitumen specimens. It asks for one day in the mould in ambient temperature, to simulate one day curing condition in the field in dry condition (Ruckel et al., 1983). Ambient temperature, between 7 to 28 days, was also used by other researchers to cure foamed asphalt mixes specimens (Long & Theyse, 2002; Nataatmadja, 2001; Saleh, 2004).

Relative humidity is hard to control in ambient temperature, and curing would impair the repeatability and reproducibility of the procedure. On the other hand, researches have shown that an average rise in the temperature enables us to control the effect of the humidity (Fu et al., 2009). When pressure is 2.3 kPa, water saturates at 20 °C and when pressure is 7.4 kPa, water saturates at 40 °C (Fredlund & Rahardjo, 1993). 10% of relative humidity at 20°C corresponds to 3% relative humidity values at 40°C. Also, 90% of relative humidity at 20°C corresponds to 28% relative humidity at 40 °C. Saying that, bigger relative humidity change span (90% - 10% = 80%) in 40°C, comparing it to smaller humidity change span in 20°C (28% - 3% = 25%), enables us to control the humidity in 40°C much easier than 20°C. Therefore, 40°C is a very acceptable temperature to practice he curing in the lab (Fu et al., 2009).

2.7.3 Humidity

As specimens are placed in the curing chamber, relative humidity (RH) would rise immediately and it takes days for it to get back to the initially set RH point in the chamber. The ascend and descend of RH is driven by the ratio of volume of the specimens to volume of the chamber. To this end, bigger chamber or smaller specimens lead to smaller fluctuation of RH (Serfass et al., 2004). Further investigation has shown that the return to the initial RH point in the chamber would happen finally and the time depends on the volume of the specimens. It could be between 7 to 14 days for big specimens. 50% RH at 40°C is proposed as an acceptable humidity for curing at the lab and it represents the field in 1 to 3 years after compaction. However, it takes months for RH to equilibrate in the field (Serfass et al., 2004).

2.8 Mechanical Properties

The best results of strength parameters must be searched in some intermediate binder content. Marshal stability used to be the most valued parameter in finding the optimal binder content as well as lowest strength loss in soaked condition (K. J. Jenkins, 2000; Muthen, 1998). The main objectives of using foamed bitumen treatment are to decrease moisture susceptibility,

heightening the fatigue resistance and improving the cohesion of the untreated aggregates to an acceptable level (Muthen, 1998). However, foamed binder content could be chosen at some other values which meets the expectations and it could be rather than the optimum amount (Muthen, 1998).

2.8.1 Moisture sensitivity

Failure in foamed asphalt mixture structures are mainly due to moisture (Iwański & Chomicz-Kowalska, 2013) and strength characteristics of the foamed mixes are highly moisture-dependent (Muthen, 1998). Moreover, moisture vulnerability of foamed mixed are important because of following considerations (K. J. Jenkins, 2000):

- Large aggregates are not coated fully with the bitumen,
- The binder content in foamed mixes is not as much as it is in HMA,
- Excessive moist on the aggregate hamper binder particles to attach to the aggregate,
- Air void content is normally high in these mixtures.

Low binder contents and high void contents in the foamed mixes make the strength characteristics moisture-dependent. Castedo (Castedo-Franco et al., 1984) found that lime could play a positive role in decreasing moisture susceptibility of the foamed mixes. Cement could play the same role in a cheaper way (Lewis & Collings, 1999). It happens because lime or cement helps the moisture-vulnerable aggregate to be coated by the binder and it also improves the compaction which leads to lower air void and higher densities (Muthen, 1998).

2.8.2 Temperature sensitivity

Since foam mix strength comes mainly from aggregates interlock, temperature can not affect it considerably (Sakr & Manke, 1985). (Bissada, 1987) proved it by comparing two equivalent foamed and hot mix group of samples in temperatures above 30 °C , which both had been cured at ambient temperature for 21 days and showed that resilient modulus change in the foamed mix samples were lower than the hot mixes. This is justifiable due to the bitumen content in

foamed mixes is not as high as it is in hot mixes, thus, not all the aggregate, particularly large ones, is covered with bitumen in foamed mixes, so the inner friction between the aggregate would not be impaired in higher temperatures in contrast to hot mixes. However, fines-bitumen mortar viscosity is reduced by increasing the temperature and it counts for loss of strength in foamed asphalt mixes (Muthen, 1998).

2.9 Summary

In this chapter you can read many research attempts which tried to address different issues about cold mixtures and particularly, foam mixes. Firstly, the concept of cold recycling described, and cold mixture variability explained along with the definitions. After that, cold mixtures structure and ingredients adequately explained and volumetric properties are discussed subsequently, and it is tried to highlight the importance of it. Mix design consideration is explored due to its role in different phases of production. Compaction as a significant part of pavement, and cold recycling specifically, is being investigated. Curing, as the most important part of this study, is fully detailed through different scientific endeavours and it is tried to come up with temperature and acceptable relative humidity (RH) as a reference point. Finally, engineering properties and cold mixtures and their sensitivities discussed.

Any of reviewed researches is a remarkable work. However, there are still lots of studies to understand cold mixes comprehensively. On the other hand, curing is hard to find investigated along with other independent variables. To this end, in this thesis, it is proposed to compare different curing methods while other independent variables such as compaction and RAP sources are considered.

CHAPTER 3

MATERIALS AND METHODOLOGY

This chapter is the description of the materials which are used in the experiments of this study and explanations of their characteristics before their utilization. It is also trying to talk about standards and definitions. It explicates the parts which may seem confusing in the production process.

Variable investigation criteria and a great number of experiments in this thesis might cause a confusion. To remove the confusion, Figure 3.1 depicts the holistic structure of the experiments. In this figure, different tests which are applied on different groups of the specimens are shown.

All mixes are produced using foamed bitumen and two different RAP sources:

- RAP1,
- RAP2.

And different compactions are applied, including:

- Marshall hammer,
- SGC 100 mm (Shear Gyratory Compactor),
- SGC 150 mm,
- proctor rammer.

Due to material quantity restrictions, RAP2 is just used in the batches 6 and 7 which compacted with Marshall hammer and SGC 100 mm respectively. Moreover, specimens of each batch are grouped in two different groups that each of which has different curing condition:

- Free surface drying (FSD),
- Partially Confined (PC)

Additionally, since SCB test requires specimens with 150 mm diameter, so this test is run just for SGC 150 mm and specimens compacted with proctor rammer which its mould is 150 mm. All other tests are applied for all the specimens.

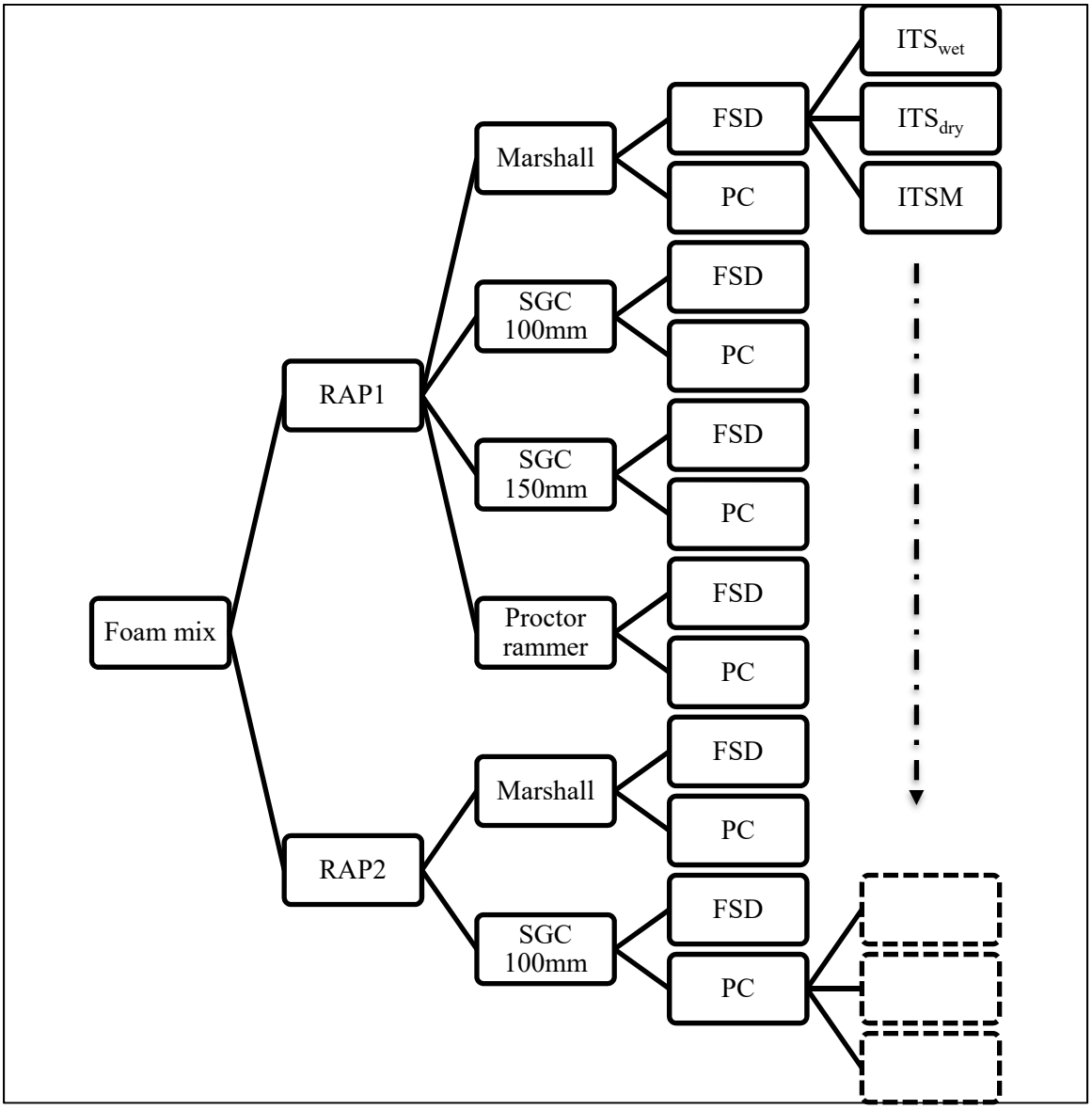

Figure 3.1 Experiments structure

3.1 Experiments Plan

Table 3.1 is the plan for experiments in this thesis:

Table 3.1 Experiments plan

RAP1			
Compaction:	**Diameter of the mould:**	**Tests**	
		FSD	**PC**
Marshall	**100 mm**	• **ITSM/ITS$_{DRY}$ (3 rep)** **ITS$_{WET}$ (3 rep)**	• **ITSM/ITS$_{DRY}$ (3 rep)** **ITS$_{WET}$ (3 rep)**
SGC	**100 mm**	• **ITSM/ITS$_{DRY}$ (3 rep)** • **ITS$_{WET}$ (3 rep)**	• **ITSM/ITS$_{DRY}$ (3 rep)** • **ITS$_{WET}$ (3 rep)**
SGC	**150 mm**	• **ITSM/ITS$_{DRY}$ (3 rep)** • **ITS$_{WET}$ (3 rep)** • **SCB (4 rep)**	• **ITSM/ITS$_{DRY}$ (3 rep)** • **ITS$_{WET}$ (3 rep)** • **SCB (4 rep)**
Proctor	**150 mm**	• **ITSM/ITS$_{DRY}$ (3 rep)** • **ITS$_{WET}$ (3 rep)1.18** • **SCB (4 rep)**	• **ITSM/ITS$_{DRY}$ (3 rep)** • **ITS$_{WET}$ (3 rep)** **SCB (4rep)**
RAP2			
Compaction:	**Diameter of the mould:**	**Tests**	
		FSD	**PC**
Marshall	**100 mm**	• **ITSM/ITS$_{DRY}$ (3 rep)** • **ITS$_{WET}$ (3 rep)**	• **ITSM/ITS$_{DRY}$ (3 rep)** • **ITS$_{WET}$ (3 rep)**
SGC	**100 mm**	• **ITSM/ITS$_{DRY}$ (3 rep)** • **ITS$_{WET}$ (3 rep)**	• **ITSM/ITS$_{DRY}$ (3 rep)** • **ITS$_{WET}$ (3 rep)**

In this table, each colour represents one batch and each batch is made in one day. Since it uses a dual shaft mixer which has capacity about 20 to 50 kg, so the size of the batch is required to be big enough to render the proper mixing. Accordingly, a small pile of dry RAP is used in each batch production, which weights approximately 30 kg and other ingredients weight accordingly.

3.2 Materials and mixtures production

After drew up the plan (Table 3.1), required material are provided as they are fully explained in following sections.

3.2.1 RAP

Structural body of all the specimens stands up mainly on the RAP, with zero amount of virgin aggregate. Two sources of RAP are available. The RAP which comes from the Republic of Saint Marino, and named RAP1, and RAP2 which comes from the United States of America. According to ASTM D448-03 standard, sieving analysis are achieved (i.e. nominal maximum particle size and gradation), Figure 3.2 and Figure 3.3. Bitumen content is characterized according to ASTM D6307 for each of the RAPs. Doing so, following characteristics are identified; Table 3.2.

Table 3.2 characteristics of the RAPs

Properties	RAP1	RAP2
Nominal maximum particle size	16 mm	10 mm
Bitumen content	5.51%	5.49%
Maximum specific gravity	2.482	2.498
Water absorption	1.1 %	1.1 %
Fragmentation @ 5 °C (fraction 5/10) - PCS	7.6 %	-
Fragmentation @ 20 °C (fraction 5/10) - PCS	6.7 %	13.9 %

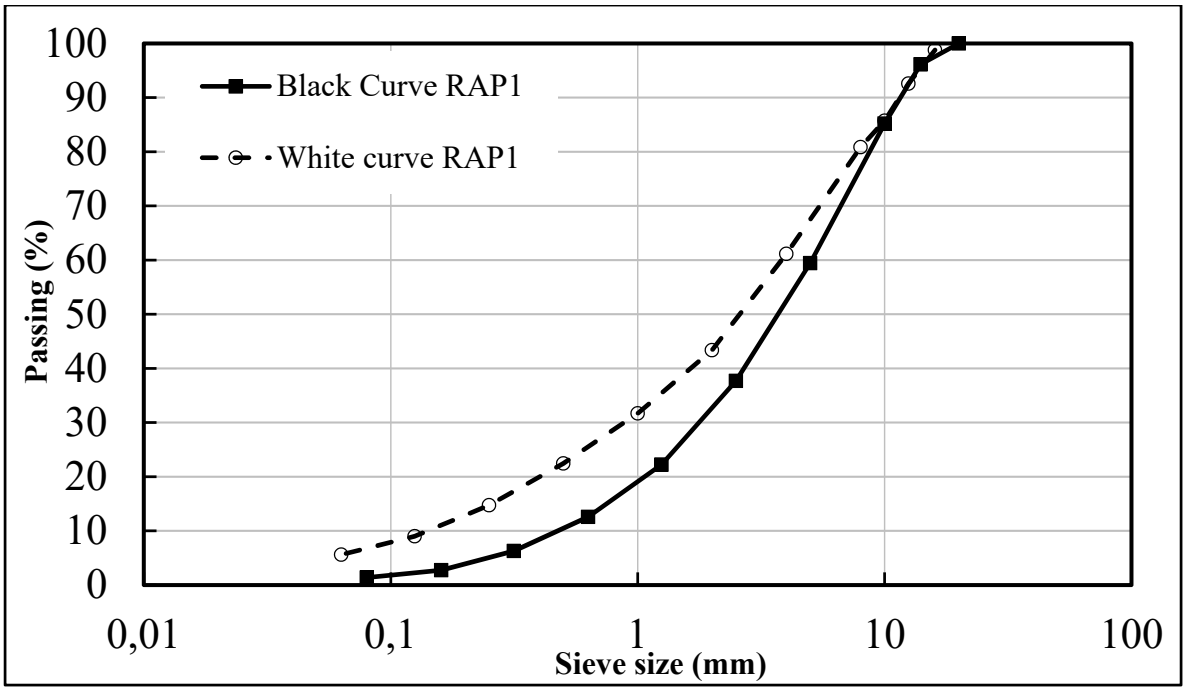

Figure 3.2 RAP1 gradation curve

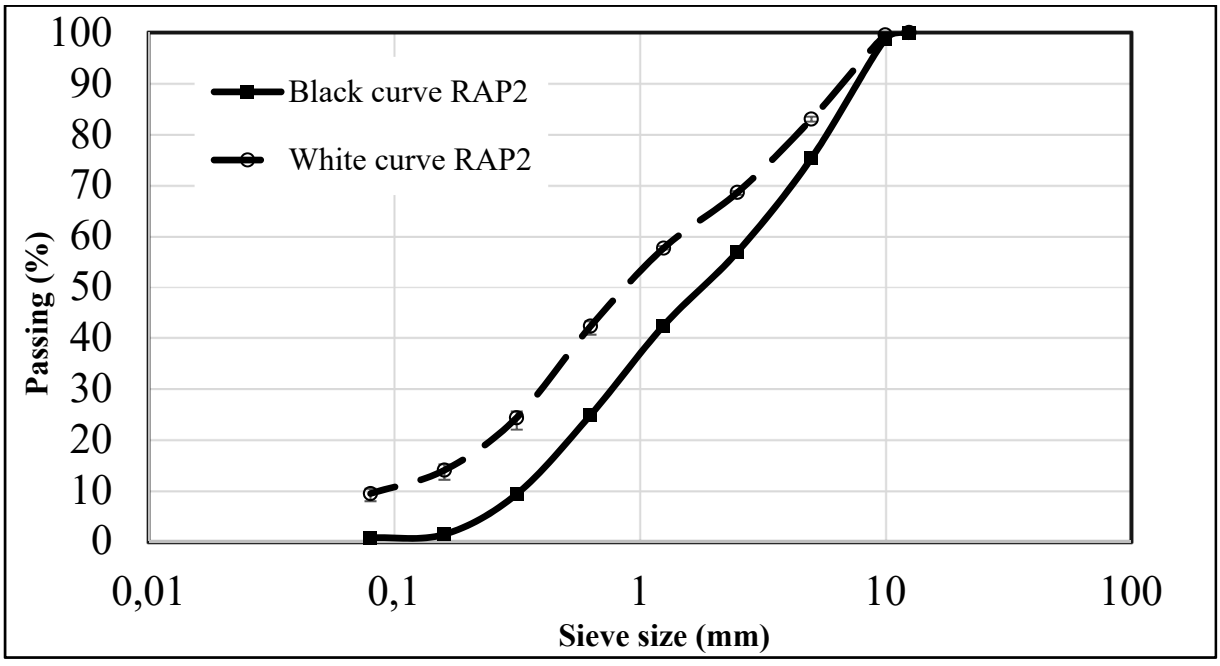

Figure 3.3 RAP2 gradation curve

3.2.2 Filler

A mineral filler (virgin aggregate) characterized by 91% of passing material at 0.008 mm sieve size, 50.1% voids of dry compacted filler (measured via Rigden apparatus), was selected to adjust both RAP gradations to follow their respective maximum density curves.

3.2.3 Bitumen

Bitumen, which is used to produce the foam in this study, has 64 $mm \cdot 10^{-1}$ value of penetration (EN 1426) and the softening point of 52 °C (EN 1427). Previous tests have shown 10, as the expansion ratio, and 6 second as the half-life (in testing with 1.9% of water by bitumen weight).

3.2.4 Cement

Cement is a GU type (CSA A3000) with compressive strength at 28 days of 43.9 MPa (ASTM C109) and Blaine surface area of 410 cm^2/g, was chosen to improve the bituminous mastic consistency and short-term resistance.

3.3 Mix design

Table 3.3 shows mix design recipe utilized in all batches production.

Table 3.3 Mix design recipe

Ingredients	Percentage according to dry RAP weight	Percentage according to solid weight (RAP+Cement+Filler)
Water	4.2%	3.88%
Filler	6.4%	5.93%
Bitumen	3.2%	2.96%
Cement	1.6%	1.48%

The RAP was put in the oven for at least 48 hours before mixing starts, Figure 3.4. Water and bitumen content are chosen based on the common practice of cold recycled materials.

Figure 3.4 RAP drying in the chamber

3.4 Apparatuses

Mixer in this study is a dual horizontal shaft, capacity of 20-50 kg, adjustable rotation speed between 60 and 190 rpm, evacuation through a lower hatch over the entire vessel length and entrance hatch for the nozzle of the foaming machine. The two shafts have 20 paddles, of which 8 are inclined in the backwards direction, and placed two by two on 5 arms.

Foamed asphalt was injected directly onto the mix with a custom built (RAP, cement, filler, water). The transfer of heat from the foam at over 100 °C, to the aggregate which is at less than 30 °C, influences the rate of collapse of the foam (i.e. the rate of viscosity increasing of the binder during the mixing). To this end, the aggregate temperature was controlled at an ambient temperature of the laboratory which is 25±1 °C.

Moreover, mixing time should be in accordance with the time required by the bitumen foam to collapse. In the laboratory, a mixing time of 60 s is applied, which is longer than in situ mixing, but simulates the difference in the energy of the laboratory mixer and field plant mixer. To this end, a mixing time of 60 s was adopted in this study.

Wirtgen WLB 10 is used in order to foam the bitumen. Following sequences are practiced every time:

- Heat the bitumen in the kettle of the foaming unit in laboratory with the pump circulating the bitumen through the system until the required temperature is achieved (normally starting with 170° C). This temperature is maintained for at least 10 minutes before foaming starts;
- Set the water-flow-meter to achieve the required water injection rate;
- Following standard procedures of the machine and calibrate the discharge rate of the bitumen and set the timer to have the required amount of foam in each try. To do so, the timer on the machine is set at 3 and 5 seconds and the weight of produced foam is measured each time respectively. Extrapolation is used and by knowing the amount of required bitumen from the mix design, and the time is set on the machine according to that;
- Couple the Wirtgen WLB10-S to the mixer, Figure 3.5, so that the foamed bitumen can be discharged directly into the mixing chamber;
- Start the mixer and allow it to mix (RAP, filler, water and cement) for at least 10 seconds before discharging the required mass of the foamed bitumen into the mixer.

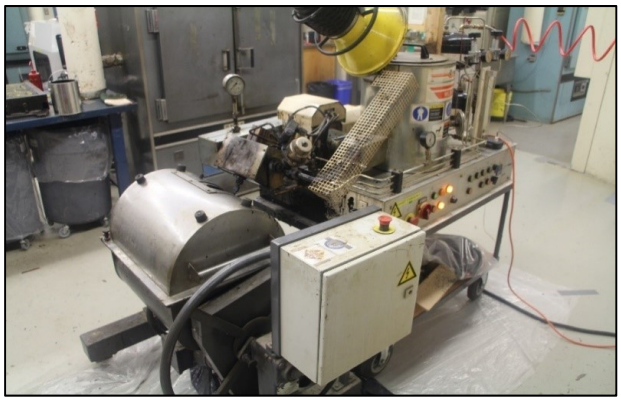

Figure 3.5 Foaming unit

3.5 Specimen production

Figure 3.6 is the flowchart to produce each batch.

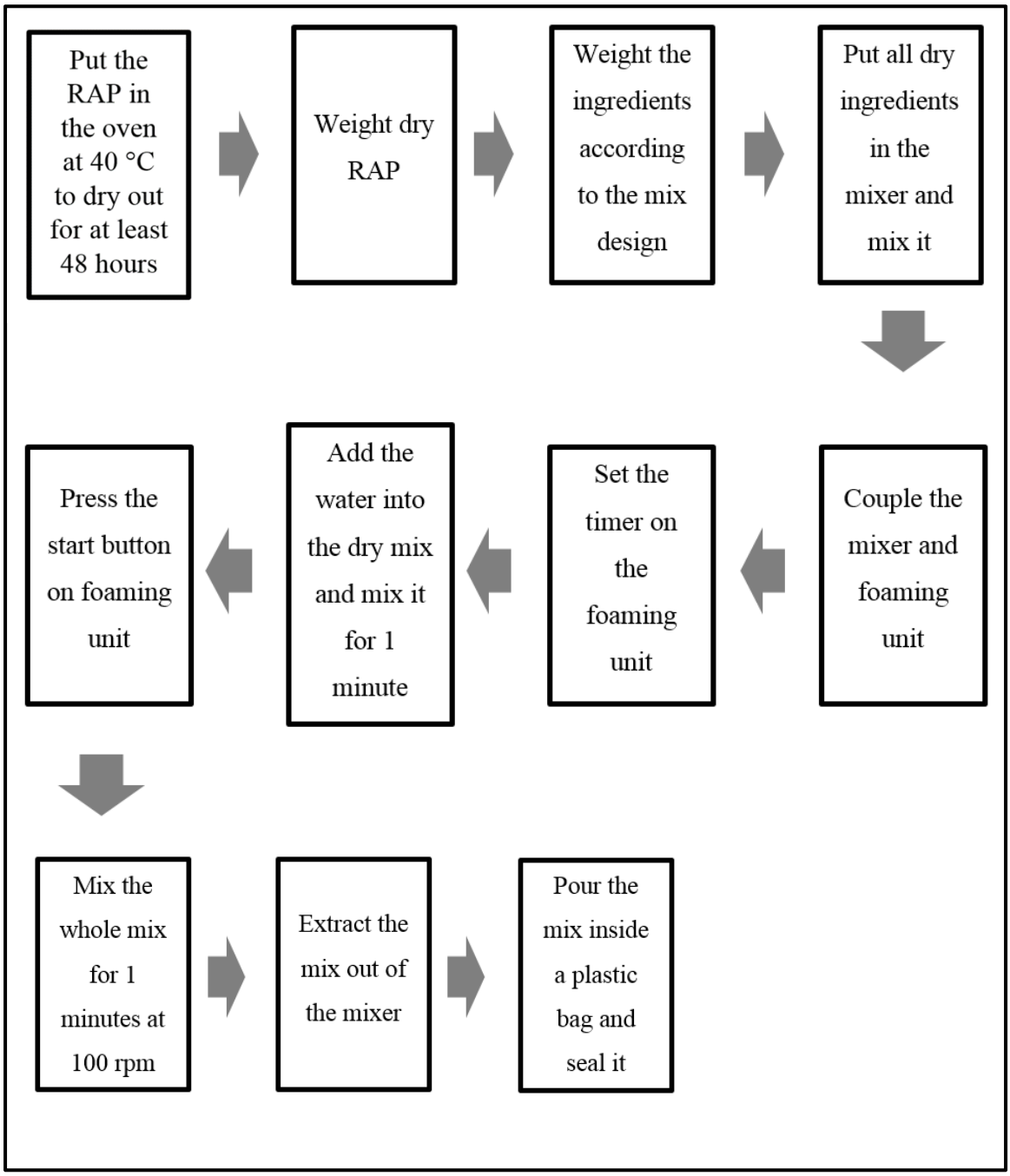

Figure 3.6 Batch production flowchart

3.5.1 Compaction methods

After having the mix ready in a sealed plastic bag, compaction would be the next step of the process in making the specimens. To do so, as it was mentioned previously, four different compaction methods are used:

- Marshall hammer,
- SGC (100 mm),
- SGC (150 mm),
- Proctor rammer.

For Marshall hammer, according to the standard, we had 50 blows on each side of the specimen and it resulted in 14.3% of air voids for the specimens in the preliminary study. For Proctor rammer compaction, similar to Marshall hammer, we don't have control neither on the energy of the compaction nor on the volume of the specimens accurately.

Moreover, as it was mentioned before, water presence in the cold mixture structure and gradual water loss during the curing period, require us to have different volumetric approach than the traditional one for HMA. It means that we need to have the volumetric properties based on the volume rather than the mass. Therefore, to have control on the volumetric properties, we need to control the volume. However, since the volumetric properties are not the objectives of this study, we don't highlight the volumetric calculations.

3.5.1.1 Marshall Hammer

Marshall hammer compactor, Figure 3.7, is an equipment to compact the mix and fabricate the specimens in order to evaluate the air void content, characteristics and performance tests. Marshall compaction conforms the standard UNI EN-12697-30. It contains different part and specifications as follows:

- It uses gravity force and falling weight energy to compact the loose material. The weight of the falling hammer is 4540g ± 10g and the height of the falling is 457 ± 3 mm. So, there is no control on the compaction energy in this method,
- Collar, base and the central part are other parts of this equipment. The base is the most down part of it which holds loose material. The collar is placed at the top of the central part just to keep everything fixed. Central part is the part that material is in it and is compacted inside.

According to the standard, 50 blows on each side of the specimen are applied. To do so, fix the central part and base. Pour the mix inside the mould in required amount which is 1100 gr in this case. Then put a paper on top of the mix and place the collar on top of the central part and locate it under the hammer. Set the blow counter at 50 and after 50 blows on the first side, put the upside of the specimen down and repeat 50 blows. Then demould the specimens using demoulding jack.

Figure 3.7 Marshall hammer

Table 3.4 Shows the Marshall hammer specifications:

Table 3.4 Marshall hammer specifications

Diameter	Height	Weight	Number of Blows
100 ± 2 mm	457.2 ± 1.5 mm	4.54 kg	50 on each side

3.5.1.2 Shear Gyratory Compactor (SGC)

Gyratory compaction approach, Figure 3.8, is firstly used to simulate the compaction of different layers of the pavement under the roller during the compaction, and secondly, to imitate the traffic loading of the pavement.

Figure 3.8 Shear Gyratory Compactor

The most important feature of Shear Gyratory Compactors SGC compaction is the improvement of interlock between the particles in the mix. SGC works to a specified angle of gyration of 1,25° under a compressive pressure of 600 kPa with compaction applying at 30 gyrations per minute. In addition, the SGC provides the facility for the measurement of the specimen height for each gyration. By giving the mix properties such as Maximum Theoretical Relative Density and the mass of the mix in the compaction mould, the air voids can be monitored during the compaction. Diameter could be either 100, or 150 mm. Cylindrical plates on the top and bottom of the mould are placed to distribute the load evenly. After putting the bottom plate, calculated amount of the mix (to reach the required air void content) is poured into the mould. Table 3.5 shows SGC specifications.

Table 3.5 SGC specifications

Central axis angle by the vertical axis	1.25°
Upper plate pressure	600 kPa
Spin rate	30 rpm

Targeting 14.3% ± 1% of air void content in the SGC (100 and 150 mm) specimens, required different height of the specimens for 100 and 150 mm specimens. For SGC specimens, since the diameter of the specimens are constant (100 and 150 mm), for controlling the volume of the specimens, and accordingly, volume of the air voids, we just need to control the height of the specimens. To address this point, the height of the specimen which is set on the gyratory compactor are presented in the Table3.6.

Table 3.6 SGC specimens' specifications

Batch number	**Mold diameter**	**RAP source**	**Height of the specimen**	**Air void (%)**	**Weight of the mix in mold before compaction**	**Range of number of gyrations**
2	100 mm	RAP1	65 mm	13.97	1059 gr	35-66
3	150 mm	RAP1	70 mm	14.00	2565 gr	52-66
4	150 mm	RAP1	70 mm	14.00	2565 gr	53-68
6	100 mm	RAP2	65 mm	13.99	1079 gr	32-58

3.5.1.3 Proctor rammer

Modified Proctor density is used worldwide to represent the field compaction. To this end, in this case, modified Proctor compaction (BS EN 13286-2: 2004) is utilized, Figure 3.9. Conforming to BS EN 13286-2 (Table 3.6), the Proctor rammer compaction is practiced.

Table 3.7 Proctor rammer specifications

Mould diameter	150.0 ± 1.0 mm
Mould height	120.0 ± 1.0 mm
Base plate thickness	150.0 – 0.5 mm
Rammer weight	4.5 ± 0.04 Kg
Number of layers	5
Number of blows in each layer	56
Height of falling rammer	457 ± 3 mm
Diameter of the base	50.0 ± 0.5 mm

Figure 3.9 Proctor rammer

Differences between the required dimensions for the tests and Proctor specimens, necessitates cutting and trimming for the Proctor specimens. The height of the proctor specimens is considered 120 mm. To conform ITS and ITSM test standards, these specimens need to get cut in half to have the least acceptable dimension which the results would be comparable with other SGC 150 mm specimens. To do so, we used the extension collar potential to produce a higher specimen than normal, which would be almost 150 mm, just to have a safety factor for cutting process, which is prone to loosing material, Figure 3.10.

Figure 3.10 Proctor rammer specimen production

3.5.2 Curing

The main research objective of this study is to evaluate a curing method which is an endeavour to simulate the field condition more accurately. As it is already stated, the main problem is to imitate the existing condition in the field which is believed doesn't let the water to move out of the specimen lateral faces in curing period (i.e. water leaves out of the specimen only through the top and bottom sides of the specimens), Figure 3.11.

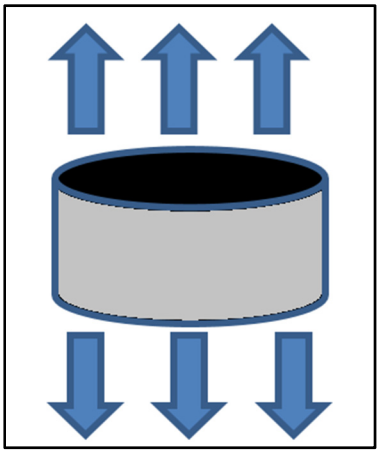

Figure 3.11 Water loss of a field extracted specimen

To evaluate this hypothesis, specimens branch out to two categories to compare the traditional curing approaches with the proposed method in this research:

- Free Surface Drying (FSD),
- Partially Confined (PC).

For example, if one batch gives 20 specimens, 10 of those are cured with FSD protocol and the other 10 are cured with PC protocol. After applying these two curing protocols on the specimens, they are put in the curing chamber at 40 ± 2 °C and 55 ± 5% humidity for 14 days.

3.5.2.1 Free Surface Drying (FSD)

In Free Surface Drying, or the conventional existing curing protocol, which is used to practice accelerated curing process in the lab, after compaction and demoulding, the specimens are introduced to the direct air and humidity condition in the chamber without any boundary between, Figure 3.12.

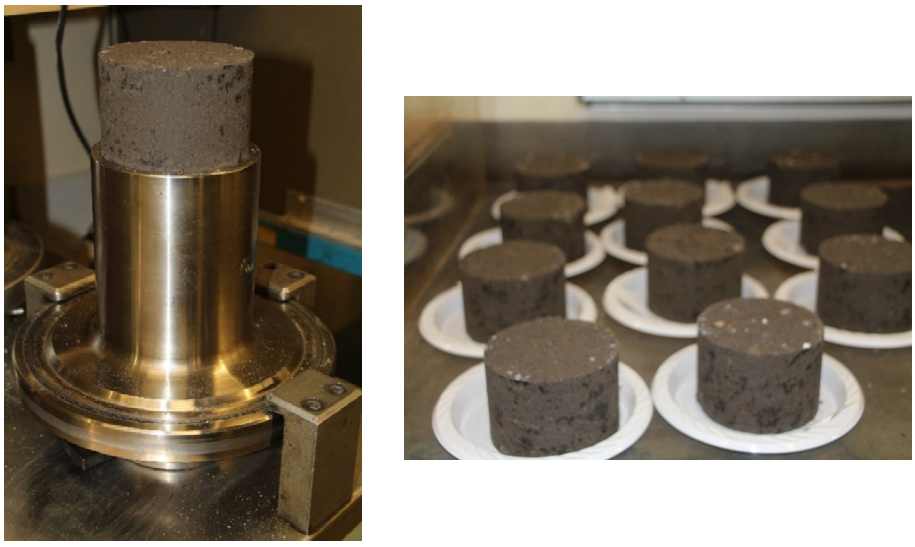

Figure 3.12 FSD curing

3.5.2.2 Partially Sealed Drying (PSD)

In the new protocol that proposed in this research, it is tried to cover the lateral side of the specimens to prevent water to leave from this part of the specimens. To do so, after mixing, compacting and demoulding the specimens, they are covered by 6 layers of cellophane, and then the top and the bottom sides of the wrapping cellophane were removed precisely, Figure 3.13. All PC specimens are put in the same chamber with the same conditions as of FSD specimens in order to have comparable conditions and conclusions.

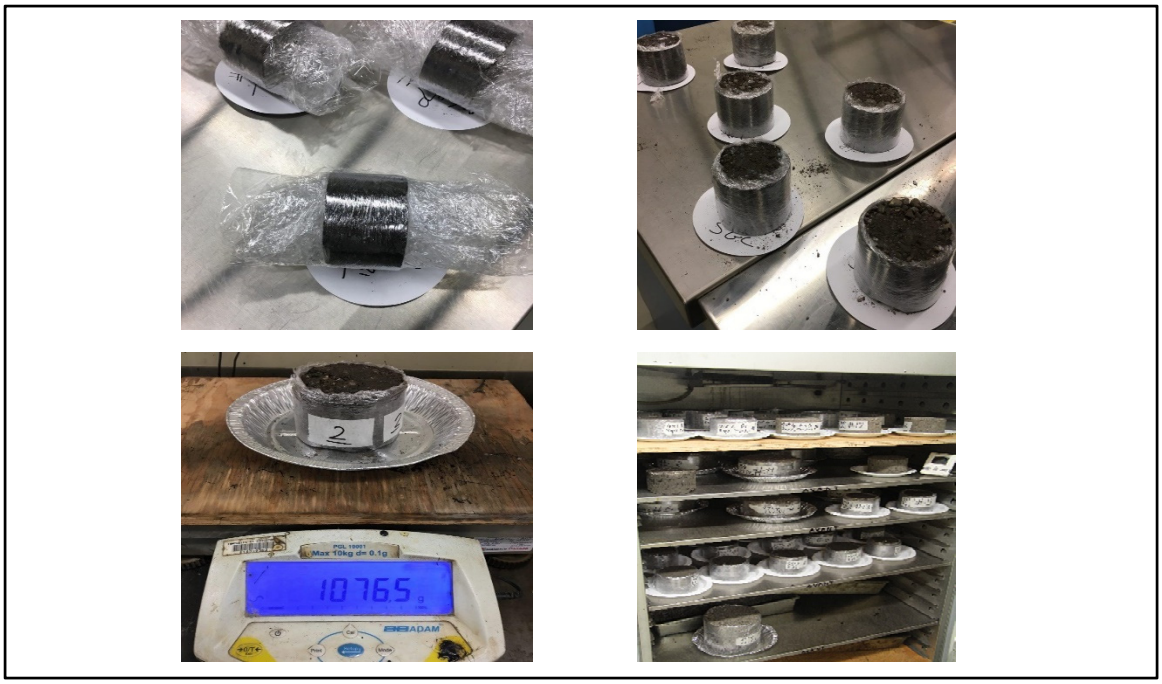

Figure 3.13 PC curing

Specimens weights are monitored in everyday of the curing period. After 14 days of curing, specimens are put out of the curing chamber, sealed and tagged in a plastic bag and kept at the room temperature until exercising the tests, Figure 3.14.

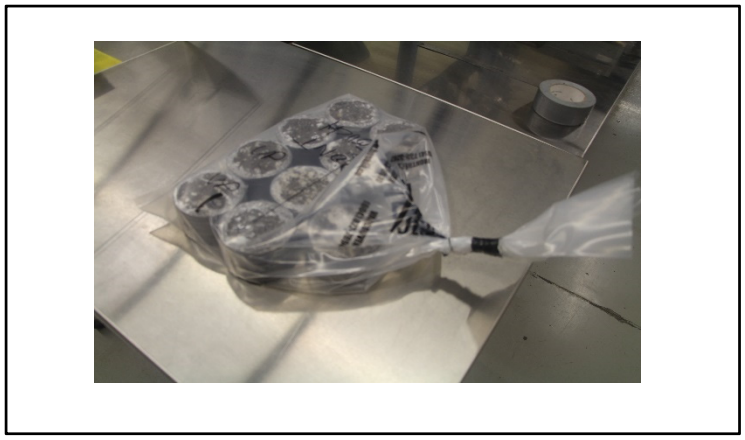

Figure 3.14 Specimens sealing before the tests

CHAPTER 4

TESTS AND RESULTS

In this chapter, all of the test results are presented. It is important to highlight that some numbers which come in the following result tables and figures in this chapter are average of the three test repetitions which come with standard deviation alongside. 72 ITS tests and 216 ITSM, and 16 SCB tests were run.

4.1 ITS (INDIRECT TENSILE STRENGTH)

ITS_{dry} and ITS_{wet} tests all conform UNI EN 12697-23, Figure 4.1. Water bath or air chamber are used to reach the specimens to the required test temperature. All tests are conducted on 25° C. Having 3 FSD specimens to report ITS_{dry} and 3 FSD specimens for ITS_{wet}. Same process is applied for PC specimens. Moreover, 6 compaction methods can be noticed (Marshal, SGC 100 mm, SGC 150 mm and proctor for RAP1; Marshal and SGC 100 mm for RAP2). Therefore, there are 72 ran (3×2×2×6 = 72), Table 4.1.

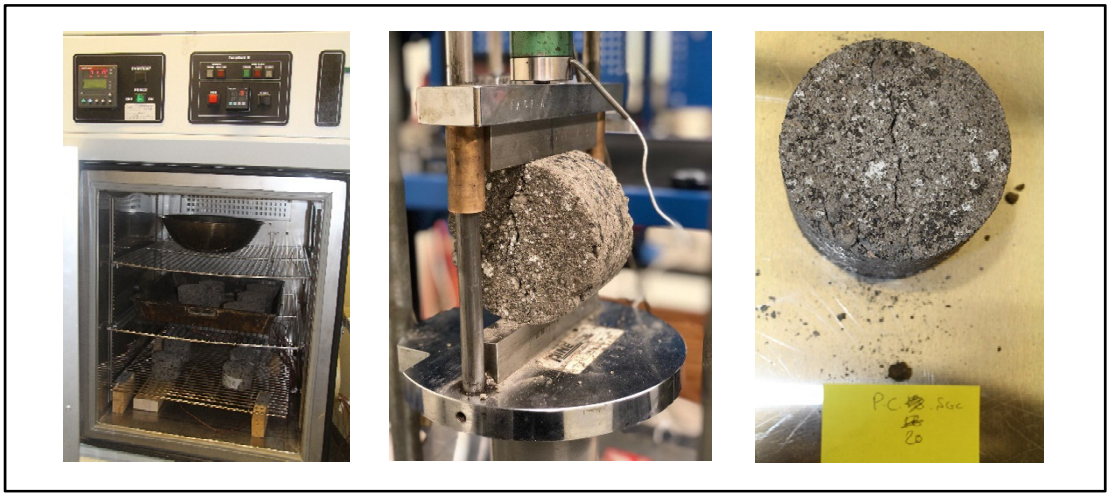

Figure 4.1 ITS test

$$\text{ITS} = \frac{2P}{\pi\, DH} \tag{4.1}$$

P = Peak load (kN)

D = Diameter (mm)

H = Height of specimen (mm)

$$\text{ITR} = \frac{\text{ITS}_{wet}}{\text{ITS}_{Dry}} \tag{4.2}$$

Table 4.1 ITS test results

BATCH NO.	COMPACTION	RAP	CURING PROTOCOL	ITS_{DRY} (kPa) avg (i=3)	SD ITS_{DRY}	ITS_{WET} (kPa) avg (i=3)	SD ITS_{WET}	ITR avg (i=3)
1	Marshall	RAP1	FSD	540.23	54.3	360.2	89.3	0.67
1	Marshall	RAP1	PC	785.91	100.3	572.7	13.4	0.73
2	SGC 100 mm	RAP1	FSD	682.94	66.4	538.8	138.9	0.79
2	SGC 100 mm	RAP1	PC	779.31	12.9	672.2	56.5	0.86
3	SGC 150 mm	RAP1	FSD	562.22	39.1	271.4	13.5	0.48
4	SGC 150 mm	RAP1	PC	684.20	61.9	685.1	63.5	1.00
5	Marshall	RAP2	FSD	512.39	37.1	431.5	22.0	0.84
5	Marshall	RAP2	PC	615.61	78.3	585.6	24.3	0.95
6	SGC 100 mm	RAP2	FSD	669.82	30.2	567.30	85.8	0.85
6	SGC 100 mm	RAP2	PC	1037.19	23.7	827.13	71.4	0.80
7	Proctor	RAP1	FSD	519.66	115.2	418.23	136.5	0.80
8	Proctor	RAP1	PC	676.04	42.6	633.89	157.6	0.94

4.2 ITSM (INDIRECT TENSILE STIFFNESS MODULOUS)

ITSM tests, Figure 4.2, all conform EN 12697-26. To this end, all specimens stayed at least 4 hours in the thermostatic chamber and test conducted at three temperatures (2 °C, 10 °C, 20 °C) and 2 perpendicular angles in each temperature for each specimen. Considering 3 specimens per each batch with the same curing conditions (FSD or PC) to do the test. 3 temperatures, 2 angles and 6 compactions (including two different diameters for SGC and two different RAP sources for Marshall and SGC 100 mm) then there are 216 tests (3×2×3×2×6 = 216), Table 4.2.

Figure 4.2 ITSM test

Table 4.2 ITSM test results

Batch	Compaction	RAP	Curing	Temperature(°C)	ITSM (MPa) avg (i=3)	SD
1	Marshall	RAP1	FSD	2	4203	540.3
				10	4396	1373.0
				20	3357	455.1
1	Marshall	RAP1	PC	2	6132	911.7
				10	5469	653.6
				20	4845	586.0
2	SGC 100mm	RAP1	FSD	2	5275	329.9
				10	3969	522.7
				20	3329	713.5
2	SGC 100mm	RAP1	PC	2	6545	883.7
				10	5469	951.7
				20	4845	535.5
3	SGC 150mm	RAP1	FSD	2	5317	738.0
				10	4251	445.9
				20	3354	295.6
4	SGC 150mm	RAP1	PC	2	8891	775.4
				10	7096	344.6
				20	5699	258.7
5	Marshall	RAP2	FSD	2	5482	1282.5
				10	4512	937.1
				20	4207	528.4
5	Marshall	RAP2	PC	2	6245	1506.7
				10	5138	1034.6
				20	6094	815.6
6	SGC 100mm	RAP2	FSD	2	6959	264.4
				10	4785	1258.8
				20	5134	388.2
6	SGC 100mm	RAP2	PC	2	8653	194.7
				10	7551	579.9
				20	6536	494.9
7	Proctor	RAP1	FSD	2	5010	425.8
				10	3932	545.1
				20	3116	289.1
8	Proctor	RAP1	PC	2	8289	2361.9
				10	5138	2024.2
				20	6094	1612.7

4.3 SCB (Semi Circular Bending)

Semi Circular Bending (SCB) tests follow UNI EN 12697-44. Figure 4.3 is the SCB test specimen shape and dimensions follow. Considering these, cutting necessitates, Figure 4.4.

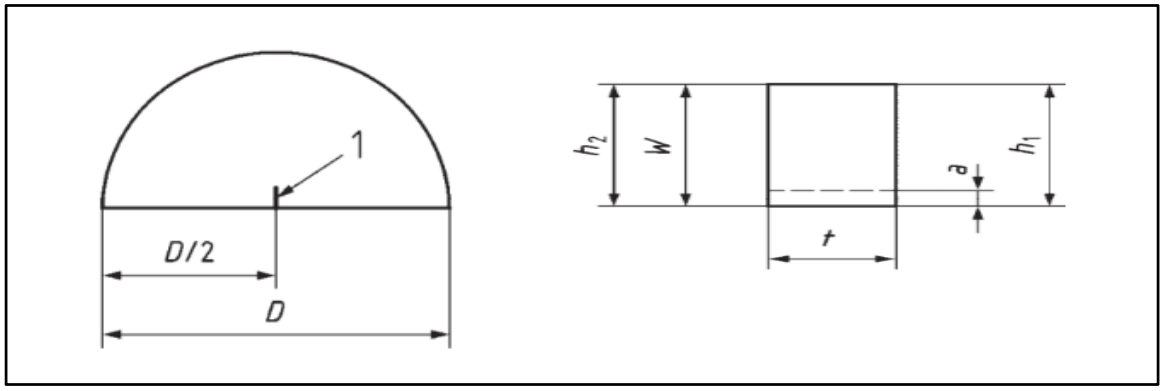

Figure 4.3 SCB specimen, shape and dimensions

Figure 4.4 Specimen trimming for SCB test

D = 150 ±1 mm

h = 50 ± 3 mm

a = (0.35 ± 0.1 mm) × (10 ± 1 mm)

There are also some values used for evaluating the results of the test as follow:

$$K_{Ic,i} = \sigma_{\max,i} \cdot f\left(\frac{a_i}{W_i}\right) \; N/mm^{\frac{3}{2}} \tag{17}$$

$$\sigma_{\max,i} = \frac{4.263 \times F_{max,i}}{D_i \times t_i} \tag{18}$$

$\sigma_{\max,i}$ = maximum stress at failure

$f\left(\frac{a_i}{W_i}\right)$ = geometric factor of the specimen

$K_{I,c}$= Fracture toughness = $\frac{\sum_{i=1}^{4} K_{Ic,i}}{4}$

F_{max} = Maximum force at failure

Failure work = Area below the force-displacement curve until maximum force, Figure 4.5.

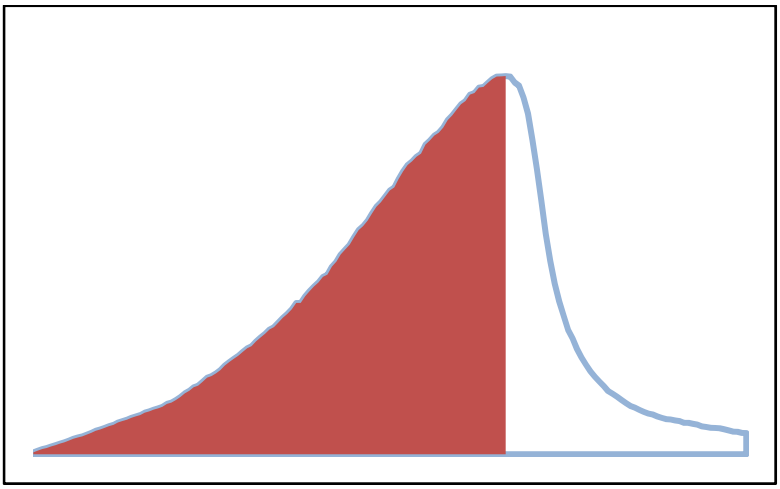

Figure 4.5 Failure work area

Table 4.3 SCB test results

Specimen	*D/2*	*t*	a	F_{max}
SGC-PC#1	75.16	53.16	9.45	1894.1
SGC-PC#2	71.24	52.17	10.45	1802.9
SGC-PC#3	74.01	51.43	9.27	2025.0
SGC-PC#4	71.87	52.03	12.08	1505.9
SGC-FSD#1	73.24	53.02	10.47	1288.9
SGC-FSD#2	74.11	48.21	9.32	597.6
SGC-FSD#3	72.55	52.89	8.95	1225.2
SGC-FSD#4	75.64	48.42	9.44	1575.6
Proctor-FSD#1	73.33	52.38	10.16	1263.1
Proctor-FSD#1	73.37	52.31	11.77	1080.6
Proctor-FSD#1	73.2	50.36	10.18	1008.2
Proctor-FSD#1	74	50.03	9.69	873.9
Proctor-PCSD#2	72.1	49.91	10.07	1111.5
Proctor-PCSD#2	74.5	50.19	10.37	1416.3
Proctor-PCSD#2	76.09	52.71	10.87	1118.4
Proctor-PCSD#2	70.28	52.29	8.47	1090.9

CHAPTER 5

RESULT ANALYSIS AND DISCUSSION

In this chapter, the results presented in chapter 4, are graphically demonstrated and analyzed. Firstly, water loss, as the curing representor, is graphically shown for different sets of specimens and analyzed via regression method. It is followed by the ITS tests, in which the different curing, RAP sources and compactions compared and discussed. ITSM results are explored in the next section and SCB results analyzes close the chapter. Following protocol in Table 5.1 is used to address and name the specimens:

Table 5.1 Specimen naming protocol

Compaction	RAP source	Curing protocol
M as **M**arshal **S** as **S**GC **P** as **P**roctor	**R1** as **R**AP**1** or **R2** as **R**AP**2**	**FSD** as **F**ree **S**urface **D**rying **PC** as **P**artially **C**onfined

For example, S100-R1-FSD refers to 100 mm diameter specimen which is compacted with SGC compactor, uses RAP1 in its structure and cured with FSD protocol.

5.1 Curing

Water weight loss was monitored during 14 days of the curing for all the specimens, considering average water loss of all specimens in the same category (specimens with the same RAP source, curing and compaction). Results are available in Figure 5.1, 5.2 and 5.3. Measurement of the exact water amount in the specimen after compaction was impossible. Because the water loss and fine particle loss in the compaction process and specimens removing and moving were inevitable and incalculable. Therefore, it is important to highlight the fact that 3.78% of water in day 0 is based only on the water content in mix preparation (Tab 3.3).

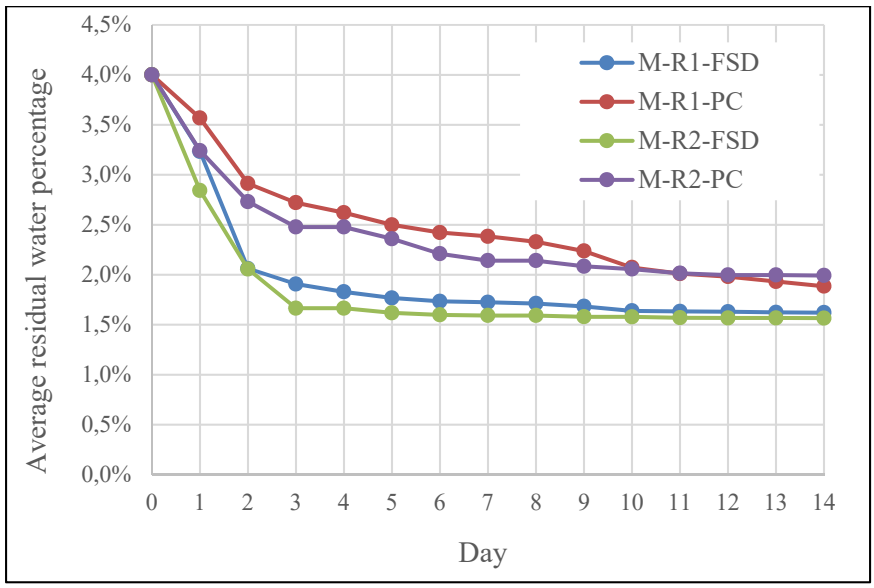

Figure 5.1 Marshall specimens water loss diagram

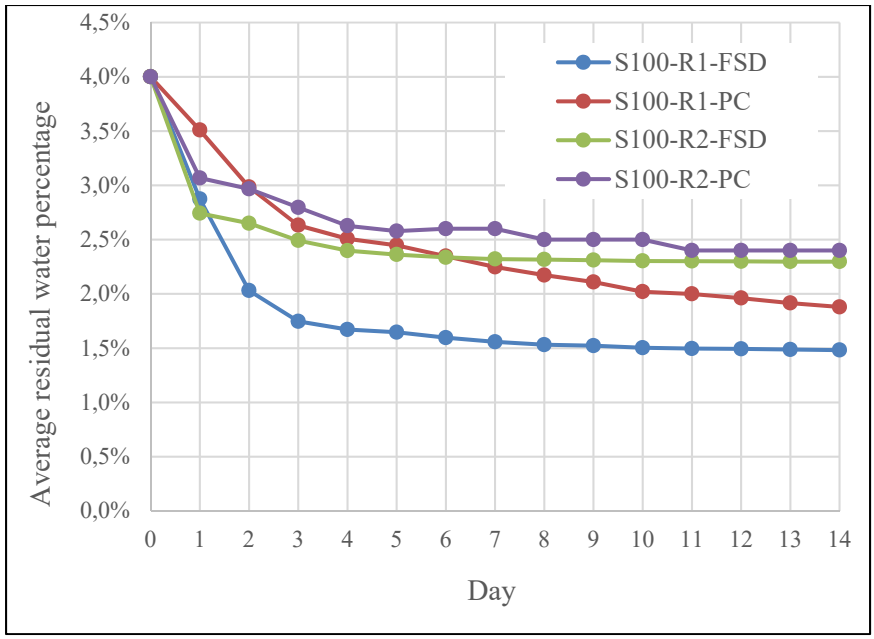

Figure 5.2 SGC specimens water loss diagram

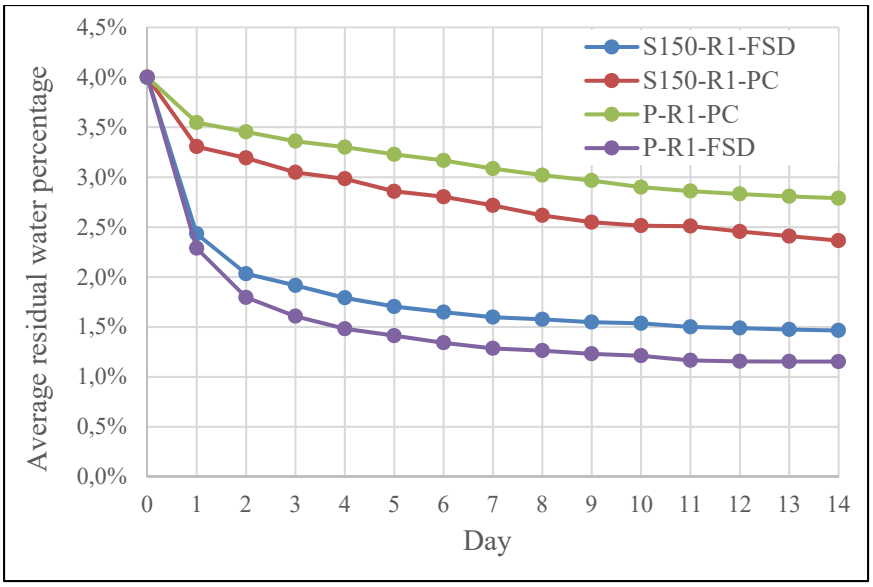

Figure 5.3 150 mm diameter specimens water loss diagram

In Table 5.2, water loss graphs are analyzed through bilinear regression analyzes. To do so, day 3 and 14 are chosen as the regression points. In Figures 5.1, 5.2 and 5.3, although FSD specimens show bigger change in the water loss rate after the first phase of water loss (day 3), however, PC specimens imply smoother change in water loss rate.

As the graphs and analyzes show, water stays longer in PC specimens. In Table 5.2, Δ_{day} is the number of the extra days in which the water content in the PC specimens would be the same as it is in FSD specimens in the day 14. For exapmle, in Table 5.2, water content in M-R1-PC specimens needs 16 more days to reach to the same level of water content as it is in M-R1-FSD at the end of the day 14. As it can be seen, the range is extended between 9 and 31 days.

Table 5.2 Bilinear regression analyzes of the water loss

	Regression factors* 0-3d (Phase 1)			Regression factors 3-14d (Phase 2)			Δ_{day} (FSD Vs PC)
Specimen	**C1**	**I_1**	**$(RI_1)^2$**	**C2**	**I_2**	**$(RI_2)^2$**	
M-R1-FSD	-0.823	3.845	0.934	-0.025	1.629	0.858	16
M-R1-PC	-0.499	3.909	0.962	-0.085	2.731	0.977	
S-R1_FSD	-0.516	3.977	0.956	-0.024	2.494	0.854	9
S-R1-PC	-0.417	4.001	0.973	-0.074	2.885	0.967	
S150-R1-FSD	-0.722	3.654	0.797	-0.033	1.834	0.906	15
S150-R1-PC	-0.336	3.844	0.823	-0.061	3.132	0.953	
P-R1-FSD	-0.834	3.623	0.825	-0.032	1.544	0.901	31
P-R1-PC	-0.254	3.935	0.839	-0.051	3.552	0.966	
M-R2-FSD	-0.856	3.724	0.952	-0.010	1.381	0.779	21
M-R2-PC	-0.563	3.798	0.951	-0.051	2.341	0.871	
S-R2-FSD	-0.507	3.525	0.739	-0.016	2.222	0.692	11
S-R2-PC	-0.406	3.635	0.786	-0.058	2.684	0.944	

* C = Slope
I = Intercept value
$(RI)^2$ = Regression coefficient

5.2 ITS

ITS test is used for material strength evaluation. In this section you can find all ITS test illustrations and analyzes. It needs to be noted that all specimen dimensions are averages of four measurements. First, ITS for specimens with different curing protocols are evaluated and it is followed by the ITS evaluation for different RAP sources and different compactions.

5.2.1 Curing

In Figure 5.4, for all specimens with the same RAP source and the same compaction, it can be deduced that in comparison to FSD, applying PC curing protocol results in stiffer specimens and it is for all sets of the specimens, Table 5.3.

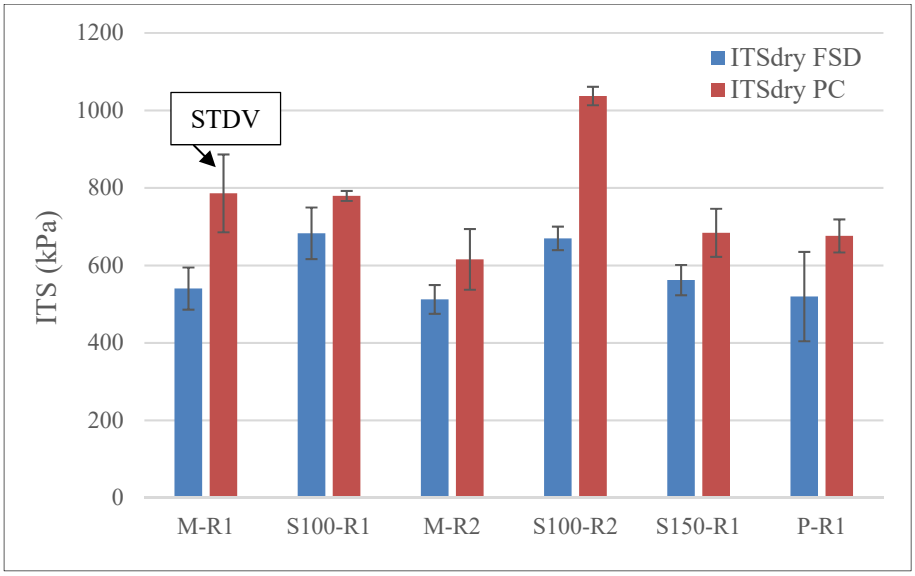

Figure 5.4 ITS results comparison for specimens with PC and FSD specimens

Table 5.3 ITS analyzes for different curing protocols

Specimen set	FSD ITS (kPa) (average, i=3)	PC ITS (kPa) (average, i=3)	Increase percentage
M-R1	540.2	785.9	45%
S100-R1	682.9	779.3	14%
M-R2	512.4	615.6	20%
S100-R2	669.8	1037.2	55%
S150-R1	562.2	684.2	22%
P-R1	519.7	676.0	30%

5.2.2 RAP sources

Figure 5.5 shows that regardless of compaction method or curing protocol, using different RAP sources (RAP1 and RAP2), doesn't change the ITS results consistently. RAP1 results in stiffer specimens in M-PC specimen sets and RAP2 results in stiffer specimens in S-PC specimen set, Table 5.4. It shows, in this study, strength changes are more curing-dependent rather than RAP-

source-dependent. On the other hand, for FSD curing these two RAP sources don't show a considerable difference in the results. It could be explained by different nature, grading and bitumen content of the aggregates of each RAP.2

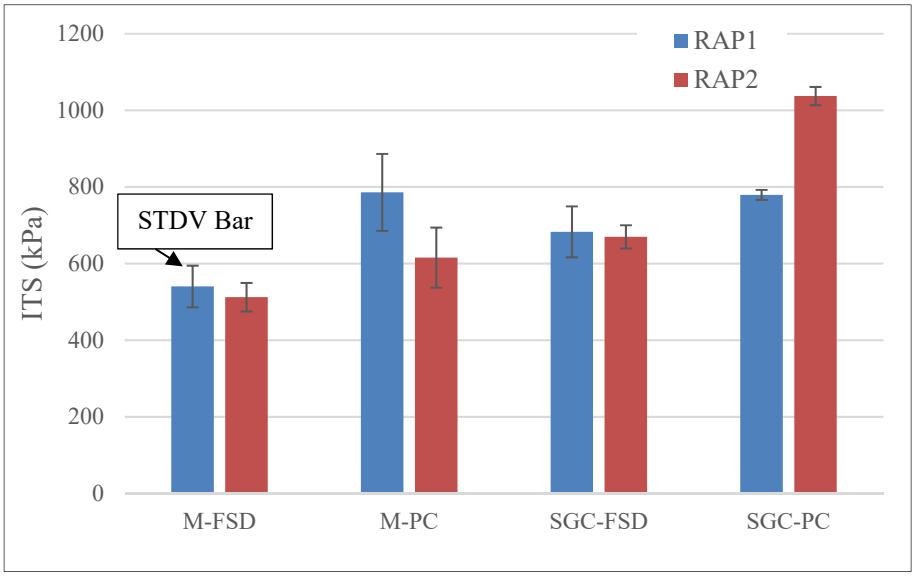

Figure 5.5 ITS results comparison for specimens with different RAP sources

Table 5.4 ITS analyzes for specimens with different RAPs

Specimen set	RAP1 ITS (kPa) (average, i=3)	RAP2 ITS (kPa) (average, i=3)	Change percentage
M-FSD	540.2	512.3	-5%
M-PC	785.9	615.6	-22%
S-FSD	682.9	669.8	-2%
S-PC	779.3	1037.1	33%

5.2.3 Compaction

Figure 5.6 shows ITS comparison for different sets of specimens with the same RAP sources and curing protocols with different compactions.

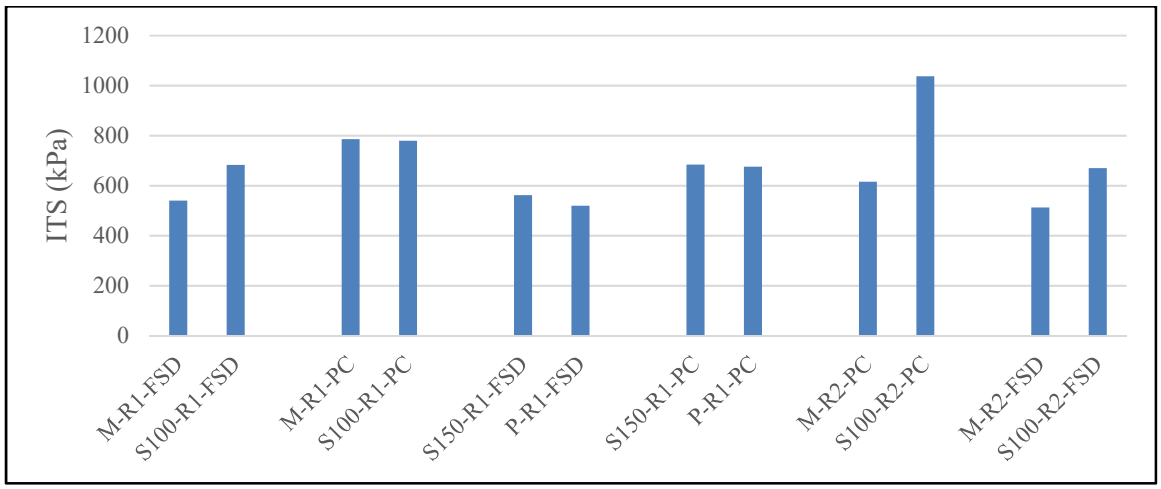

Figure 5.6 ITS results comparison for different sets of specimens

In this graph, every two specimen sets with the same curing protocols and RAP sources, and different compactions are compared. As it can be recognized, for FSD curing, SGC compaction results in stiffer specimens. For PC specimen with RAP1, there is no considerable difference between Marshall and SGC compaction. Moreover, for the specimens with RAP1, the change spans are smaller than it is in the specimens with RAP2. It shows that the stiffness of the specimens with RAP1 are less sensitive to compaction method, Table 5.5.

Table 5.5 ITS analyzes for the specimens with the same RAP and curing protocols, and different compactions

Specimen set	ITS (kPa) (average, i=3)	ITS change
M-R1-FSD	540.2	S100 = 1.26 M
S100-R1-FSD	682.9	
M-R1-PC	785.9	S100 = 1.09 M
S100-R1-PC	779.3	
S150-R1-FSD	562.2	S150 = 1.08 P
P-R1-FSD	519.6	
S150-R1-PC	684.2	S150 = 1.01 P
P-R1-PC	676.0	
M-R2-PC	615.6	S100 = 1.68 M
S100-R2-PC	1037.1	
M-R2-FSD	512.3	S100 = 1.3 M
S100-R2-FSD	669.8	

5.3 ITR

ITR, as a moisture resistance indicator (see section 4.1), is also affected by applying different curing protocols, Figure 5.7. ITR changes are not as much as the ITS changes. Although, S100-R2 lost their moisture resistance slightly; however, it shows the new curing protocol has improved ITR for the rest of the specimen sets and the most rise of the ITR is for S150-R1 specimens (Table 5.6). On the other hand, moisture resistance improvements in the big diameter specimens (S150 and P) are more than the smaller specimens.

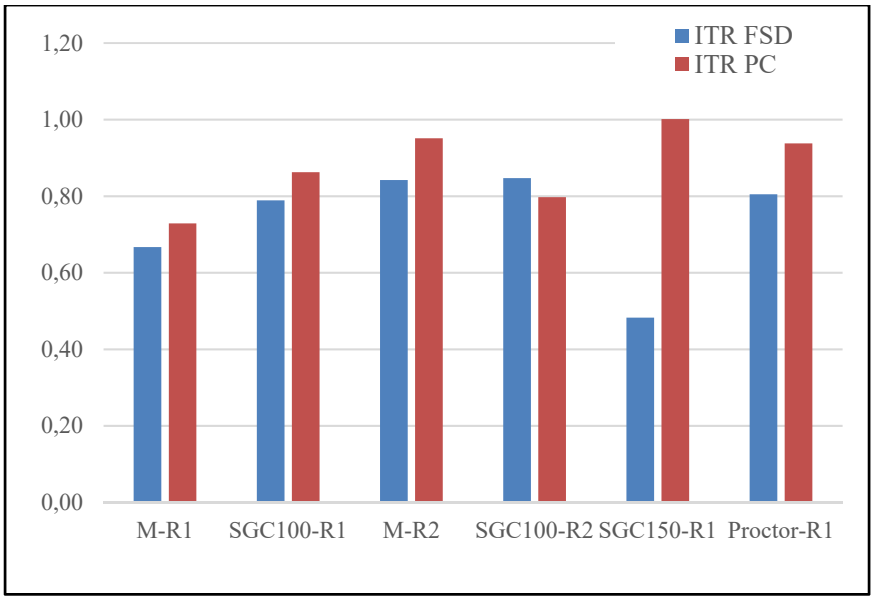

Figure 5.7 ITR results comparison for PC and FSD specimens

Table 5.6 ITR results analyzes for PC and FSD specimens

Specimen set	ITR FSD (kPa) (average, i=3)	ITR PC (kPa) (average, i=3)	Change percentage
M-R1	0.67	0.73	9%
S100-R1	0.79	0.86	9%
M-R2	0.84	0.95	13%
S100-R2	0.85	0.80	-6%
S150-R1	0.48	1.00	107%
P-R1	0.80	0.94	17%

5.4 ITSM

ITSM as another stiffness representor is also an important indication of characteristic and engineering properties. It provides us with stiffness modulus which is required for designing objectives. In this section, ITSM results are analyzed and graphically discussed for different curing, RAP sources and compactions. To this end, in each case, the evaluated criteria is

considered as the dependent variables and the other criteria are taken as the independent variables.

5.4.1 Curing

As it is shown in the Figure 5.8, applying PC curing protocol improves ITSM results in different temperatures (for M-R1 specimens), Table 5.7. It means that for two sets of specimens with the same RAP sources and compactions, PC curing improves ITSM results in all the temperatures. On the other hand, the changes of ITSM results show that by temperature rise, the ITSM decreases (as it was expected). This means a more predictable behaviour for PC curing protocol than the FSD.

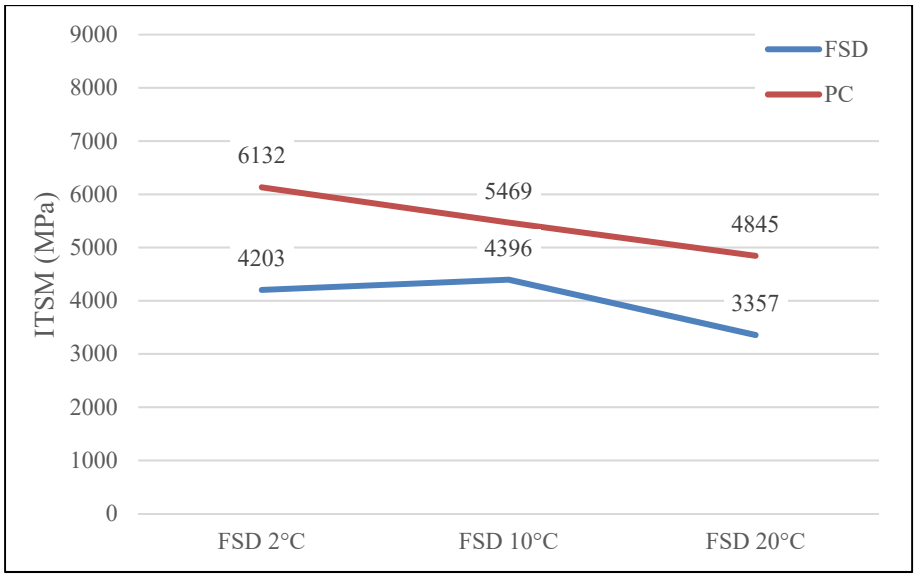

Figure 5.8 Comparison of the ITSM results in different temperatures for M-R1 specimens with different curing

Table 5.7 ITSM analyzes of the M-R1 specimens with different curing

T (°C)	FSD (average, i=3)	ITSM change for different T (FSD)*	PC (average, i=3)	ITSM change for Different T (PC)	ITSM change for different curing**
2 °C	4203	-	6132	-	46%
10 °C	4396	4.6%	5469	-10%	24%
20 °C	3357	-20%	4845	-21%	44%
* ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM (2°C) **ITSM change for different curing = (ITSM(PC) – ITSM (FSD)) / ITSM (FSD)					

For SGC compacted specimens with RAP1, PC curing improves ITSM results, Figure 5.9. According to this figure, in SGC compaction, ITSM change pattern in both FSD and PC curing are similar than the Marshall compaction (Table 5.8). To this end, they are more predictable.

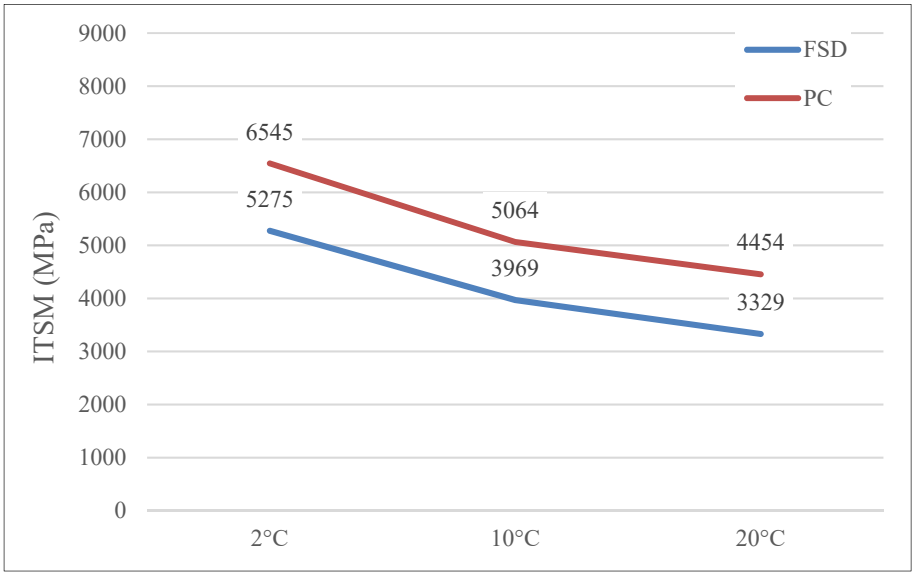

Figure 5.9 Comparison of the ITSM results in different temperatures for S100-R1 specimens with different curing

Table 5.8 ITSM analyzes of the S100-R1 specimens with different curing

T(°C)	FSD (average, i=3)	ITSM change for different T (FSD)*	PC (average, i=3)	ITSM change for different T (PC)	ITSM change for different curing**
2 °C	5275	-	6545	-	24%
10 °C	3969	-24%	5064	-22%	28%
20 °C	3329	-36%	4454	-32%	34%
* ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM (2°C) **ITSM change for different curing = (ITSM(PC) – ITSM (FSD)) / ITSM (FSD)					

For S150 specimens with RAP1, the improvements in the ITSM results are very considerable and almost twice big as S100 or Marshall compaction, Table 5.9. Adding to that, the change of ITSM for both curing follows the same pattern as S100 specimens and it makes them more foreseeable, Figure 5.10.

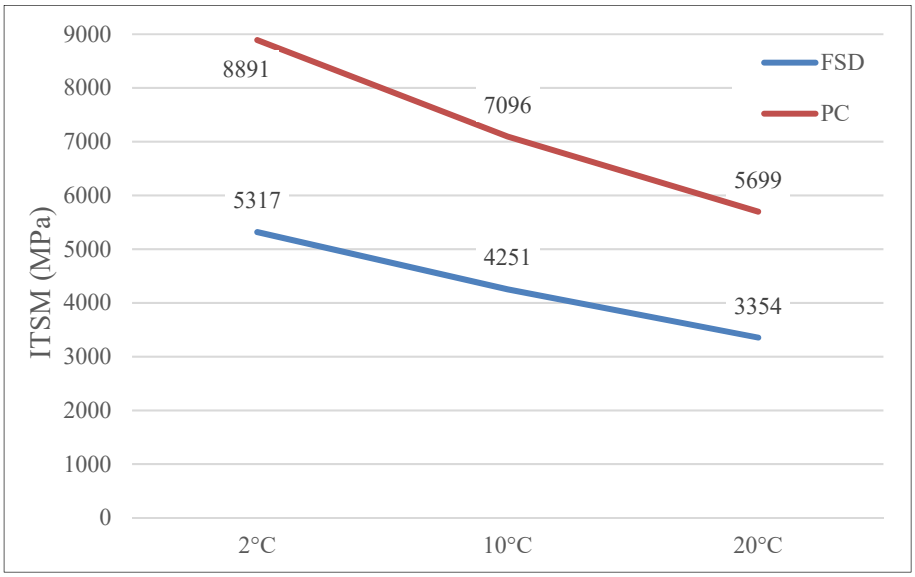

Figure 5.10 Comparison of the ITSM results in different temperatures for S150-R1 specimens with different curing

Table 5.9 ITSM analyzes of the S150-R1 specimens with different curing

T (°C)	FSD (average, i=3)	ITSM change for different T(FSD)*	PC (average, i=3)	ITSM change for Different T(PC)	ITSM change for different curing**
2 °C	5317	-	8891	-	67%
10 °C	4251	-20%	7096	-20%	67%
20 °C	3354	-37%	5699	-36%	70%
* ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM (2°C) **ITSM change for different curing = (ITSM(PC) – ITSM (FSD)) / ITSM (FSD)					

The improvement of ITSM results also happens for the Proctor compacted specimens with RAP1. The change spans (see Figure 5.11 and Table 5.10) are about the same as S150 mm (see Figure 5.10 and Table 5.9). However, the only inconsistency in the results, is the ITSM growth as temperature increases in PC specimens, Figure 5.11.

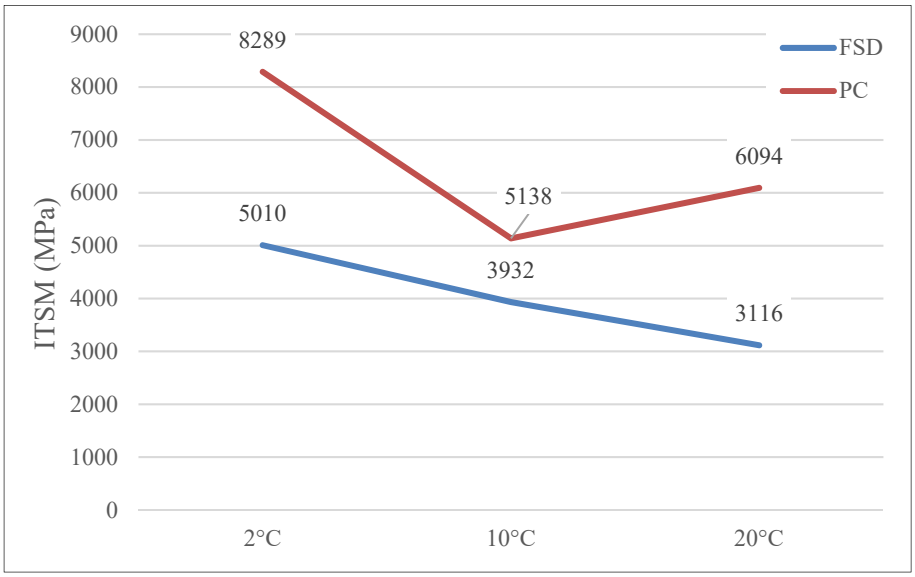

Figure 5.11 Comparison of the ITSM results in different temperatures for P-R1 specimens with different curing

Table 5.10 ITSM analyzes of the P-R1 specimens with different curing

T (°C)	FSD (average, i=3)	ITSM change for different T(FSD)*	PC (average, i=3)	ITSM change for different T (PC)	Change percentage for different curing**
2 °C	5010	-	8289	-	65%
10 °C	3932	-21%	5138	-38%	31%
20 °C	3116	-37%	6094	-26%	96%

* ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM (2°C)
**ITSM change for different curing = (ITSM(PC) – ITSM (FSD)) / ITSM (FSD)

For specimens set with the same RAP source (RAP2) and Marshall compaction, PC curing improves the ITSM results. It is notable that this increase of ITSM is not as big as it was for the M-R1 specimens. Although, ITSM for 2 and 10°C are increased equally, however, this pattern has not happened for 20°C (see Table 5.11). Contrary to the M-R1 specimens, here FSD specimens behave more consistently for RAP2 and it makes them more predictable.

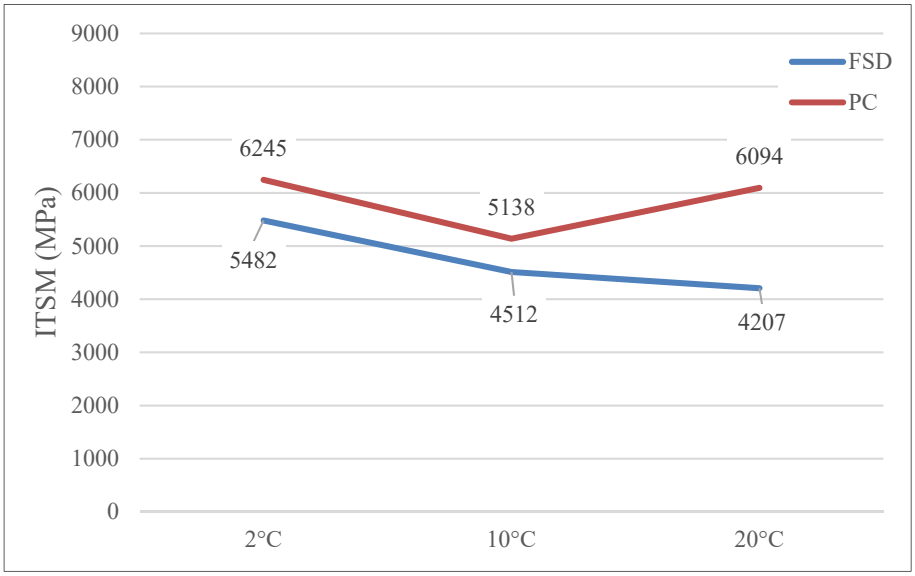

Figure 5.12 Comparison of the ITSM results in different temperatures for M-R2 specimens with different curing

Table 5.11 ITSM analyzes of the M-R2 specimens with different curing

T (°C)	FSD (average, i=3)	ITSM change for different T(FSD)*	PC (average, i=3)	ITSM change for different T(PC)	Change percentage for different curing**
2 °C	5482	-	6245	-	14%
10 °C	4512	-17%	5138	-17%	14%
20 °C	4207	-23%	6094	-2%	45%
* ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM (2°C) **ITSM change for different curing = (ITSM(PC) – ITSM (FSD)) / ITSM (FSD)					

S100-R2 specimens show a positive reaction to PC curing in the matter of stiffness change. As it is seen in Figure 5.13 and Table 5.12, PC curing enhances the stiffness of the specimens. It does not only show PC curing effects on the stiffness increase, but also it helps the behaviour to get more consistent results, which makes it more predictable. Moreover, the stiffness change ratio is almost the same as M-R1 specimens (Table 5.7 and Figure 5.8), and slightly less than specimens with SGC compaction and RAP1 (Table 5.8 and Figure 5.9).

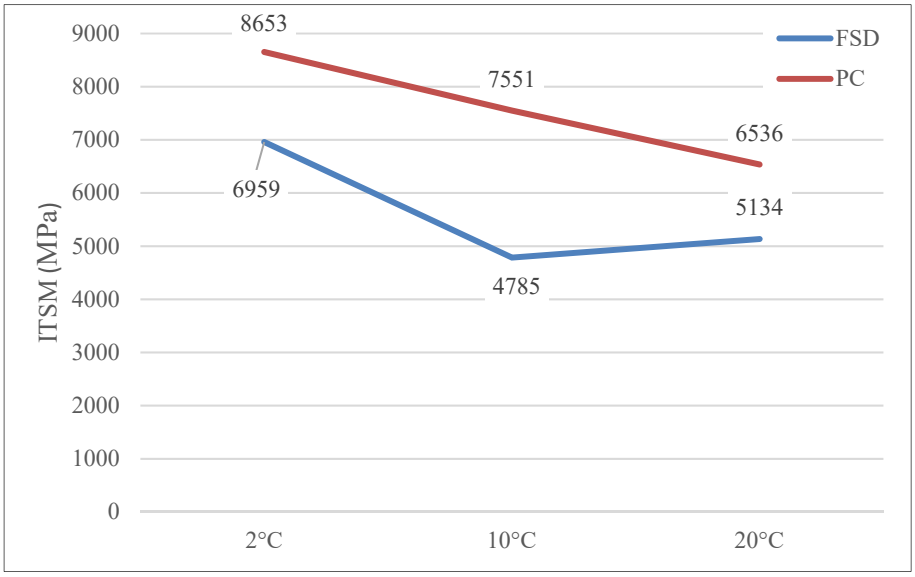

Figure 5.13 Comparison of the ITSM results in different temperatures for S100-R2 specimens with different curing

Table 5.12 ITSM analyzes of the S100-R2 specimens with different curing

T(°C)	FSD (average, i=3)	ITSM change for different T(FSD)*	PC (average, i=3)	ITSM change for different T(PC)	Change percentage for different curing**
2 °C	6959	-	8653	-	24%
10 °C	4785	-31%	7551	-13%	58%
20 °C	5134	-26%	6536	-24%	27%
* ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM (2°C) **ITSM change for different curing = (ITSM(PC) – ITSM (FSD)) / ITSM (FSD)					

Figure 5.14 is an overview of all ITSM tests for all the specimens with different curing, RAP sources and compactions in different temperatures. In the right side of this figure, the ITSM of specimens with PC curing are exhibited. Comparing it to the left side of this figure (which shows ITSM for different specimens with different RAP sources, temperatures and compaction), the role of PC curing on producing stiffer specimens can be recognized easily.

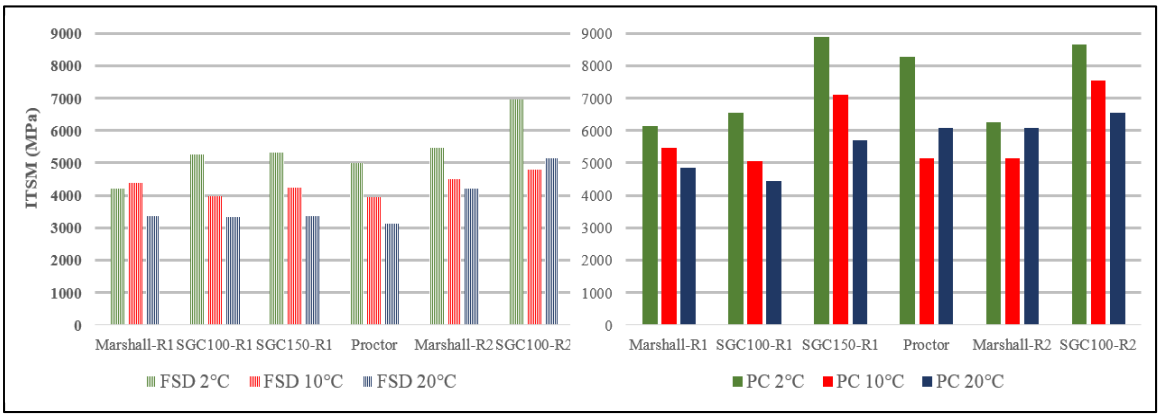

Figure 5.14 ITSM comparison for different sets of the specimens in different temperatures

5.4.2 RAP sources

In this part of the thesis, the role of RAP source on the final ITSM results are evaluated and discussed. To this end, RAP1 and RAP2 are considered as the independent variables and ITSM is considered as the dependent variable. Therefore, in each case, by having sets of specimens

with the same curing and compaction, and different RAP sources, the ITSM tests are evaluated and analyzed graphically and numerically. In Figure 5.15, ITSM for M-FSD specimens and different temperatures and RAP sources are illustrated. According to this figure, RAP2 always responses better in matter of ITSM improvement in M-FSD specimens; although, the ITSM difference between RAP1 and RAP2 in 10°C is not considerable. Moreover, specimens with RAP2 behave more expectedly with temperature rise in M-FSD set of the specimens, Table 5.13.

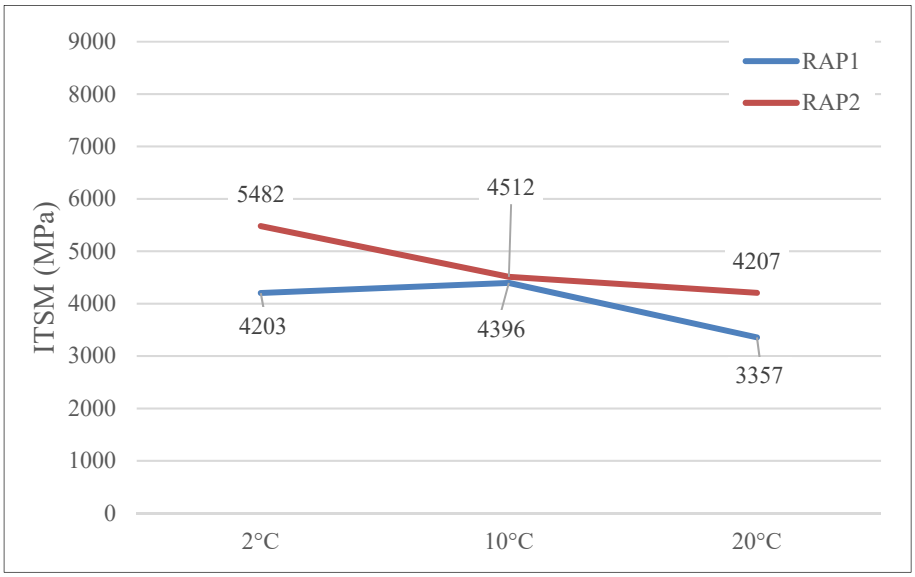

Figure 5.15 Comparison of the ITSM results in different temperatures for M-FSD specimens with different RAPs

Table 5.13 ITSM analyzes of the M-FSD specimens with different RAPs

T(°C)	R1 (average, i=3)	ITSM change for different T° (R1)*	R2 (average, i=3)	ITSM change for different T° (R2)	Change percentage for different RAPs**
2 °C	4203	-	5482	-	30%
10 °C	4396	5%	4512	-18%	3%
20 °C	3357	-20%	4207	-23%	25%
*ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM (2°C) **ITSM change for different RAPs = (ITSM(R2) – ITSM(R1)) / ITSM(R1)					

In M-PC specimens, RAP1 results in a more consistent pattern (by temperature rise, ITSM consistently decreases), Figure 5.16. Other comparisons, including different RAP sources, and the RAP2 affect on the results (with temperature rise) are inconsistent, Table 5.14.

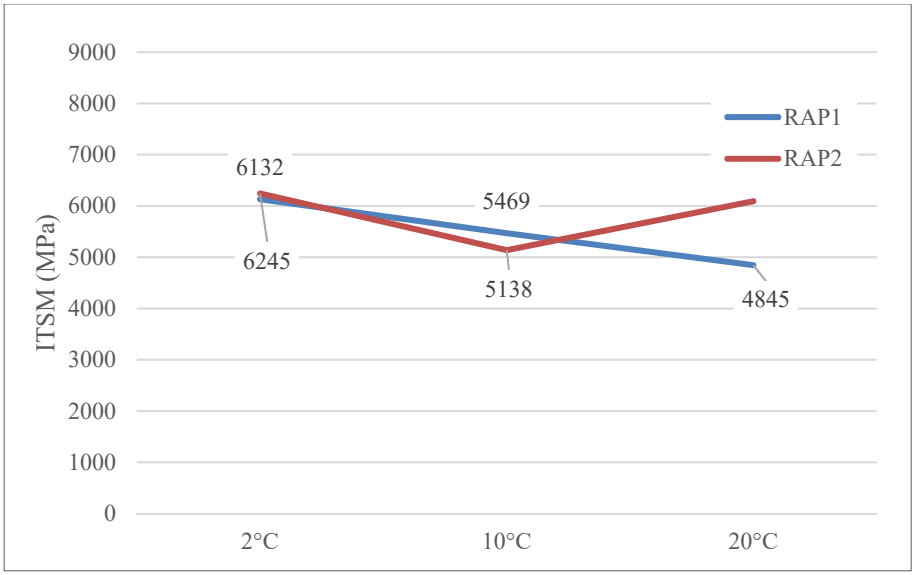

Figure 5.16 Comparison of the ITSM results in different temperatures for M-PC specimens with different RAPs

Table 5.14 ITSM analyzes of the M-PC specimens with different RAPS

T (°C)	RAP1 (average, i=3)	ITSM change for different T(R1)*	RAP2 (average, i=3)	ITSM change for different T (R2)	Change percentage for different RAPs**
2 °C	6132	-	6245	-	2%
10 °C	5469	-11%	5138	-18%	-6%
20 °C	4845	-21%	6094	-2.4%	26%
*ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM (2°C) **ITSM change for different RAPs = (ITSM(R2) – ITSM(R1)) / ITSM(R1)					

For S100-FSD specimens, RAP2 always results in stiffer specimens, Figure 5.17. Contrary to RAP2, RAP1 behaves more consistently through the temperature rise, Table 5.15.

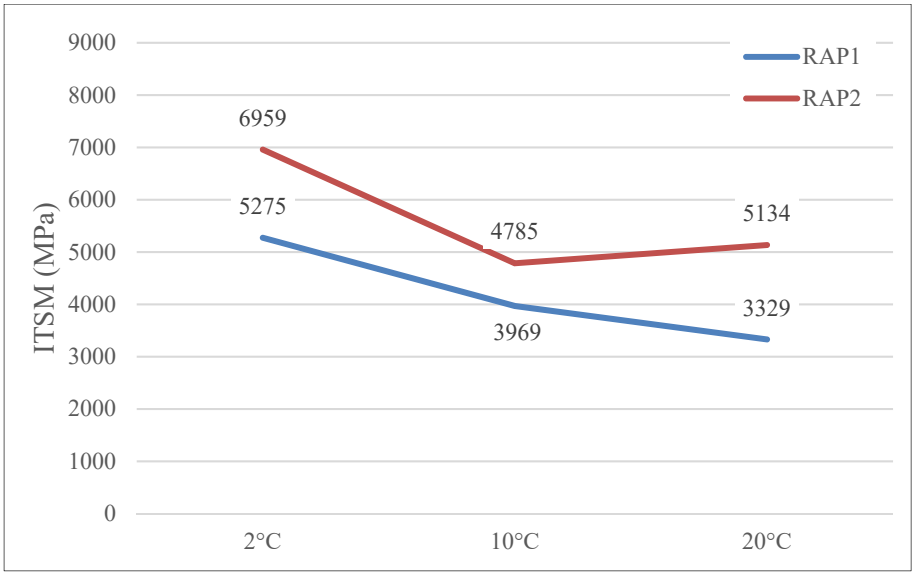

Figure 5.17 Comparison of the ITSM results in different temperatures for S100-FSD specimens with different RAPs

Table 5.15 ITSM analyzes of the S100-FSD specimens with different RAPs

T (°C)	RAP1 (average, i=3)	ITSM change for different T(R1)*	RAP2 (average, i=3)	ITSM change for different T(R2)	Change percentage for different RAPs**
2 °C	5275	-	6959	-	32%
10 °C	3969	-25%	4785	-31%	21%
20 °C	3329	-36%	5134	-26%	54%
*ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM (2°C) **ITSM change for different RAPs = (ITSM(R2) – ITSM(R1)) / ITSM(R1)					

Figure 5.18 shows that for S-PC specimens, RAP2 results in higher ITSM in all temperatures, and both RAP1 and RAP2 behave consistently, i.e. ITSM descends by temperature ascending, Table 5.16.

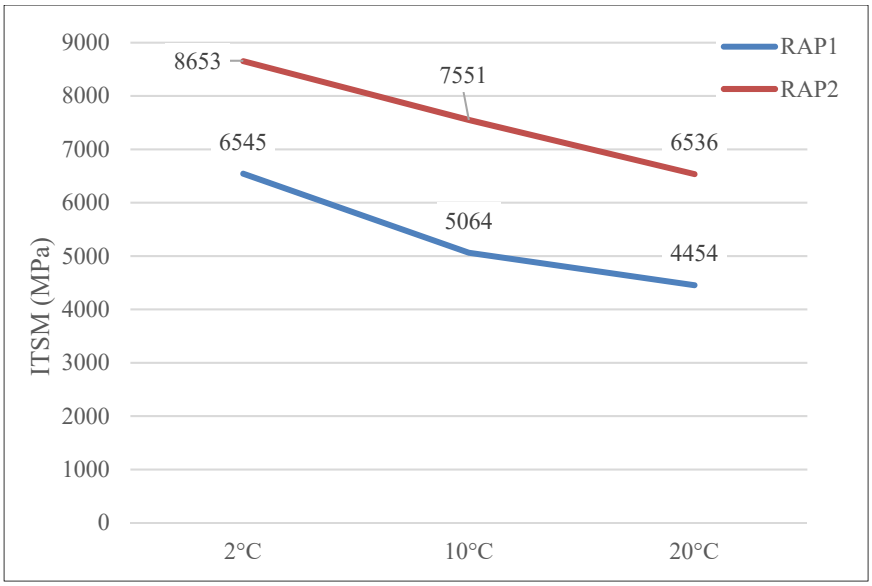

Figure 5.18 Comparison of the ITSM results in different temperatures for S100-PC specimens with different RAPs

Table 5.16 ITSM analyzes of the S100-PC specimens with different RAPs

T (°C)	RAP1 (average, i=3)	ITSM change for different T(R1)*	RAP2 (average, i=3)	ITSM change for different T(R2)	Change percentage for different RAPs**
2 °C	6545	-	8653	-	32%
10 °C	5064	-23%	7551	-13%	49%
20 °C	4454	-32%	6536	-24%	47%
*ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM (2°C) **ITSM change for different RAPs = (ITSM(R2) – ITSM(R1)) / ITSM(R1)					

In Figure 5.19, ITSM for all sets of M-FSD, M-PC, S100-FSD and S100-PC with different RAP sources are compared. Refering to this figure, S-PC and S-FSD specimens are more affected by changing the RAP source from RAP1 to RAP2, in comparison to the rest of the specimens. For all of the specimen sets and different temperatures, but M-PC at 10°C, RAP2 produce stronger specimens. Marshall compacted specimens are not affected by the RAP source considerably.

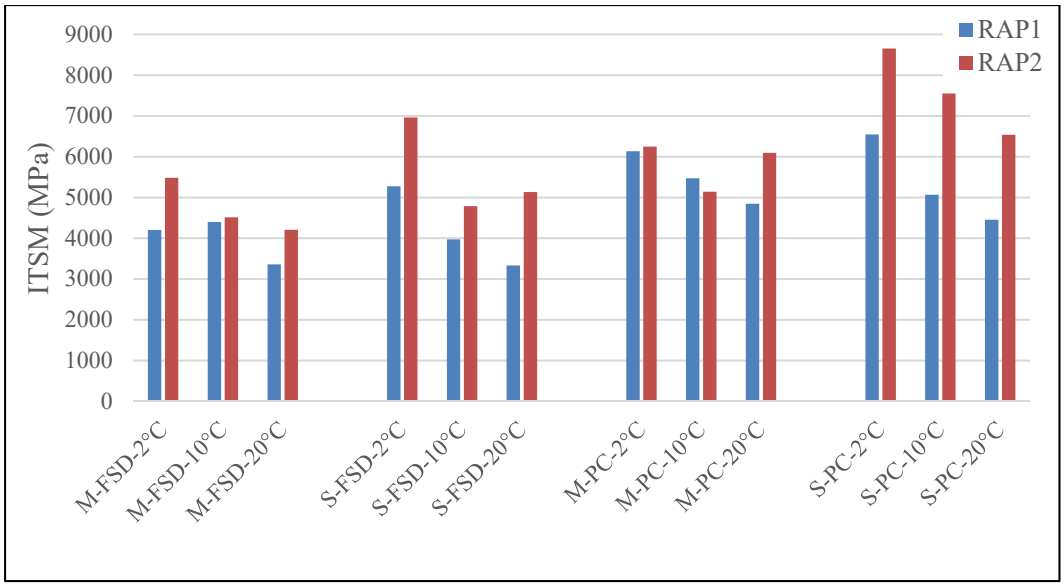

Figure 5.19 Comparison of the ITSM results in different temperatures for different sets of specimens

5.4.3 Compaction

This section tries to evaluate the effect of the compaction on the final ITSM results. To do so, like two previous sections in which targeted criteria is considered as independent variable and ITSM as dependent, here, compaction is considered as independent variable and ITSM is dependent one.

For R1-FSD specimens, the only recognizable trend from Figure 5.20 is the gradual decrease in S-R1-FSD specimens with temperature rise, Table 5.17.

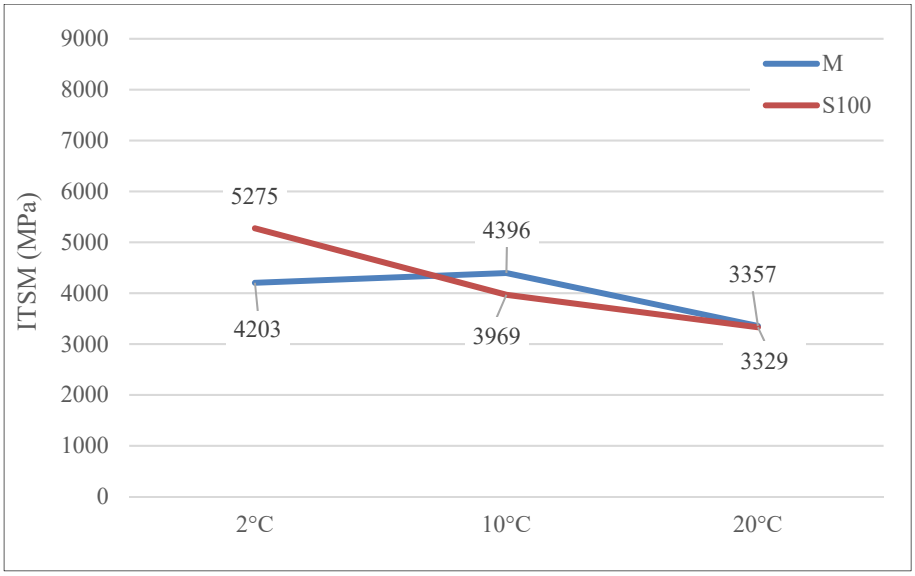

Figure 5.20 Comparison of the ITSM results in different temperatures for R1-FSD specimens with different compactions

Table 5.17 ITSM analyzes of the R1-FSD specimens with different compactions

T (°C)	M (average, i=3)	ITSM change for different T(M)*	S100 (average, i=3)	ITSM change for different T(S)	Change percentage for different compactions**
2 °C	4203	-	5275	-	25%
10 °C	4396	5%	3969	-25%	-10%
20 °C	3357	-20%	3329	-36%	-1%
*ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM(2°C) **ITSM change for different RAPs = (ITSM(S) – ITSM (M)) / ITSM(M)					

R2-FSD specimens result in stiffer specimens with SGC compaction (Figure 5.21) and Marshall compacted specimens lose their stiffness with temperature rise. However, SGC specimens stiffnesses don't follow this pattern, Table 5.18.

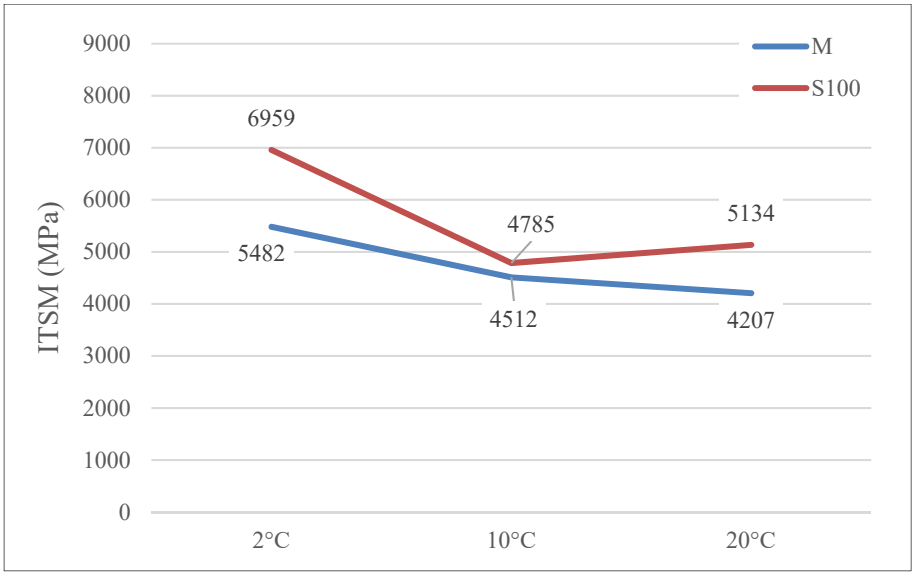

Figure 5.21 Comparison of the ITSM results in different temperatures for R2-FSD specimens with different compactions

Table 5.18 ITSM analyzes of the R2-FSD specimens with different compactions

T (°C)	M (average, i=3)	ITSM change for different T(M)*	S100 (average, i=3)	ITSM change for different T(S)	Change percentage for different compactions**
2 °C	5482	-	6959	-	27%
10 °C	4512	-18%	4785	-31%	6%
20 °C	4207	-7%	5134	7%	22%
*ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM(2°C) **ITSM change for different RAPs = (ITSM(S) – ITSM (M)) / ITSM(M)					

In lower temperature, R1-PC specimens are stiffer than when they are compacted with SGC, Figure 5.22. However, as the temperature rises, Marshall compacted specimens show bigger modulus than SGC compacted ones. Both SGC and Marshall compacted specimens behave expectedly and lose their modulus value as temperature rises, Table 5.19.

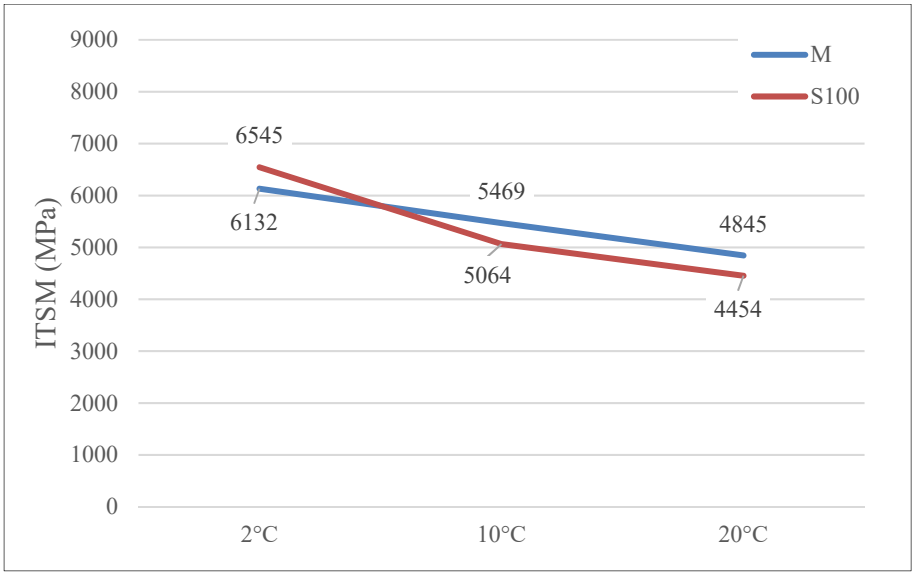

Figure 5.22 Comparison of the ITSM results in different temperatures for R1-PC specimens with different compactions

Table 5.19 ITSM analyzes of the R1-PC specimens with different compactions

T (°C)	M (average, i=3)	ITSM change for different T(M)*	S100 (average, i=3)	ITSM change for different T(S)	Change percentage for different compactions**
2 °C	6132	-	6545	-	7%
10 °C	5469	-11%	5064	-23%	-7%
20 °C	4845	-21%	4454	-32%	-8%
*ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM(2°C) **ITSM change for different RAPs = (ITSM(S) – ITSM (M)) / ITSM(M)					

Figure 5.23 and Table 5.20 show ITSM changes for R2-PC specimens with different compactions. S-R2-PC specimens are always higher in modulus value, comparing to Marshall compacted specimens (M-R2-PC). However, in higher temperature (20°C), the difference is much smaller than low temperature (2°C). Moreover, SGC compacted specimens lose their modulus as temperature rises, which is not true for the Marshall compacted specimens.

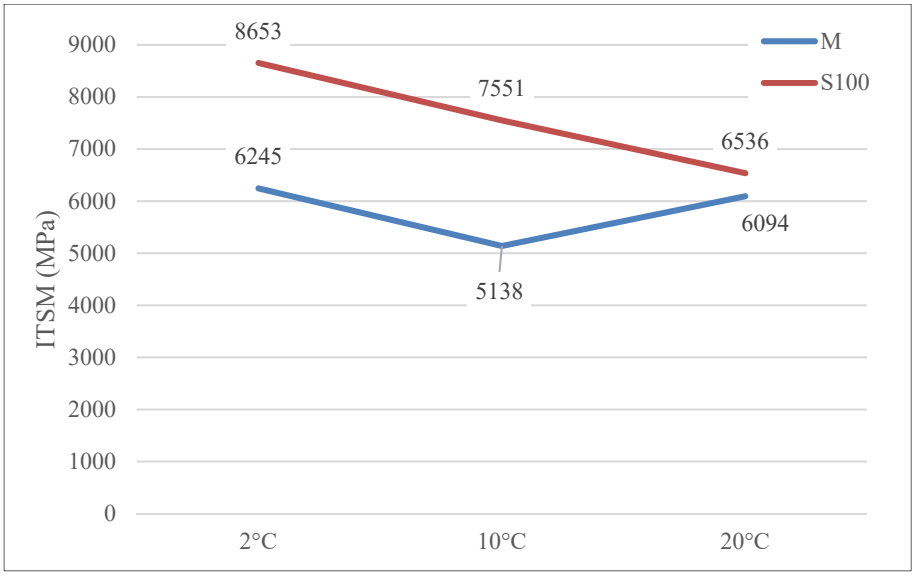

Figure 5.23 Comparison of the ITSM results in different temperatures for R2-PC specimens with different compactions

Table 5.20 ITSM analyzes of the R2-PC specimens with different compactions

T (°C)	M (average, i=3)	ITSM change for different T(M)*	S100 (average, i=3)	ITSM change for different T(S)	Change percentage for different compactions**
2 °C	6245	-	8653	-	39%
10 °C	5138	-18%	7551	-13%	47%
20 °C	6094	-2%	6536	-24%	7%
*ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM(2°C) **ITSM change for different RAPs = (ITSM(S) – ITSM (M)) / ITSM(M)					

As it is seen in Figure 5.24, for big-diameter specimens (150 mm), S-FSD specimens always yield higher modulus. However, the difference between two methods of the compaction, S150 and proctor, is not considerable in any of the temperatures and both of them lose their stiffness by temperature rise, Table 5.21.

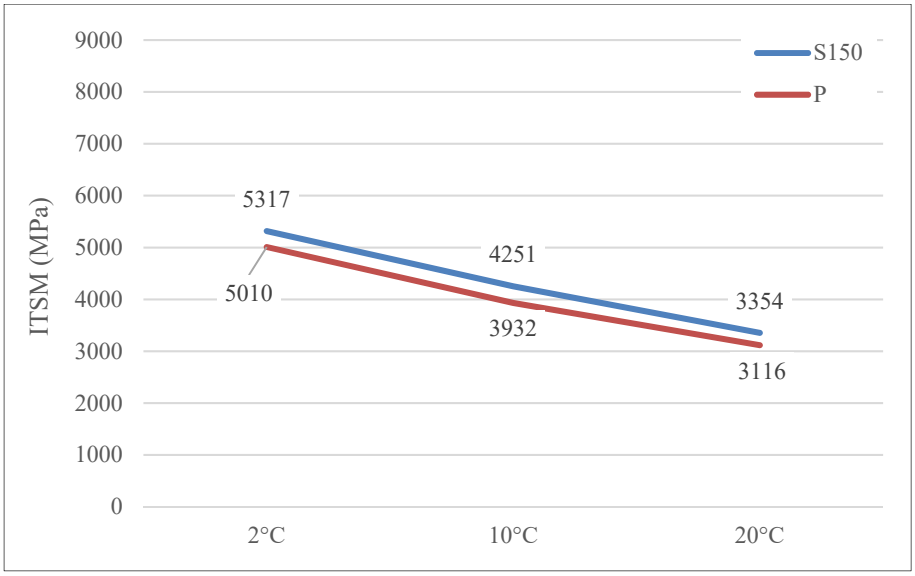

Figure 5.24 Comparison of the ITSM results in different temperatures for R1-FSD specimens (150mm) with different compactions

Table 5.21 ITSM analyzes of the R1-FSD specimens (150mm) with different compactions

T (°C)	M (average, i=3)	ITSM change for different T(M)*	S150 (average, i=3)	ITSM change for different T(S)	Change percentage for different compactions**
2 °C	5317	-	5010	-	-6%
10 °C	4251	-20%	3932	-22%	-8%
20 °C	3354	-37%	3116	-37%	-7%
*ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM(2°C) **ITSM change for different RAPs = (ITSM(S) – ITSM (M)) / ITSM(M)					

Big diameter specimens (S150 and Proctor) with RAP1 and PC curing show more stiffness with SGC compaction in 2°C and 10°C. However, it changes in higher temperature (20°C), Figure 5.25. Moreover, in 2°C and 20°C both compactions yield close results.

SGC compacted specimens, contrary to Proctor compacted ones, consistently lose their stiffness as the temperature rises.

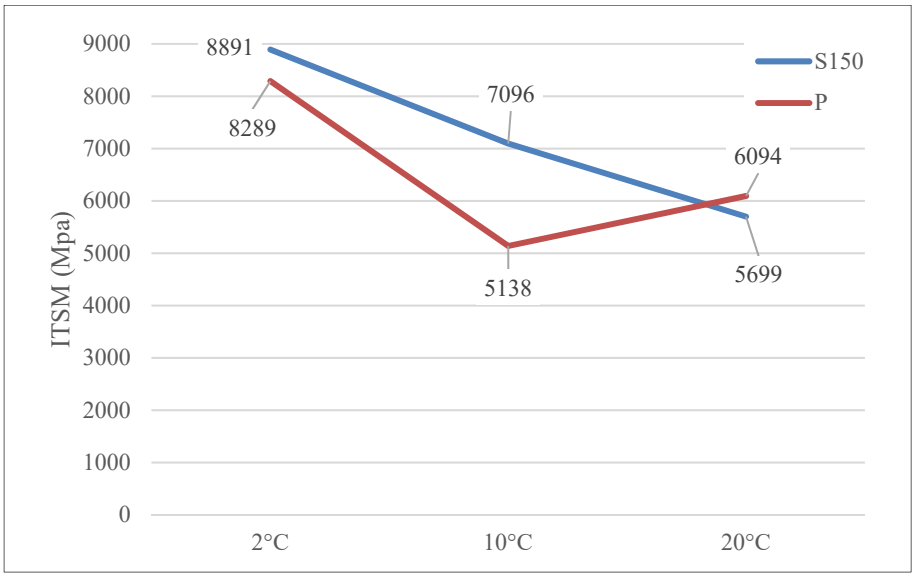

Figure 5.25 Comparison of the ITSM results in different temperatures for R1-PC specimens (150mm) with different compactions

Table 5.22 ITSM analyzes of the R1-PC specimens (150mm) with different compactions

T (°C)	M (average, i=3)	ITSM change for different T(M)*	S150 (average, i=3)	ITSM change for different T(S)	Change percentage for different compactions**
2 °C	8891	-	8289	-	-7%
10 °C	7096	-20%	5138	-38%	-28%
20 °C	5699	-36%	6094	-26%	7%
*ITSM change for different T = (ITSM(X°C) – ITSM(2°C)) / ITSM(2°C) **ITSM change for different RAPs = (ITSM(S) – ITSM (M)) / ITSM(M)					

5.5 SCB

Due to easy operation, configuration, specimen preparation, repeatability and reproducibility, SCB test is one of the considerable test methods to evaluate mix characteristics. SCB test report (see section 4.3) includes $K_{Ic,i}$ which is the fracture toughness. Final $K_{Ic,i}$ for each specimen set is an average of four $K_{Ic,i}$, Table 5.23.

Table 5.23 SCB test results analyzes

Specimen	$K_{Ic,i}$	K_{Ic}	F_{max}	$F_{max(Average)}$	**Fracture work**	**Work (Average)**
S150-PC#1	5.613	5.92	1894.1	1806.9	769.27	701.1
S150-PC#2	5.850		1802.9		656.10	
S150-PC#3	6.665		2025.0		780.32	
S150-PC#4	5.553		1505.8		598.80	
S150-FSD#1	4.219	3.81	1288.9	1171.8	542.80	445.9
S150-FSD#2	1.953		597.6		152.11	
S150-FSD#3	3.592		1225.2		-	
S150-FSD#4	5.508		1575.6		642.95	
P-FSD#1	4.094	3.46	1263.1	1056.4	897.42	511.4
P-FSD#2	3.497		1080.6		391.72	
P-FSD#3	3.408		1008.2		323.67	
P-FSD#4	2.843		873.9		432.81	
P-PC#1	3.803	3.93	1111.5	1184.3	525.33	426.5
P-PC#2	4.806		1416.3		521.66	
P-PC#3	3.592		1118.4		302.79	
P-PC#4	3.531		1090.9		356.38	

S-PC specimens show the biggest value for fracture work. In fact, for S-150 specimens, PC curing protocol application has improved the fracture resistance for 57%. However, PC curing didn't improve the cracking resistance for Proctor compacted specimens; even though, F_{max}

has been improved for P-PC specimens. Force-displacement experimental curves are presented in Figures 5.26 and 5.27. Comparing these two figures, Figure 5.30, shows SGC compaction dominance over Proctor compaction in both FSD and PC curing.

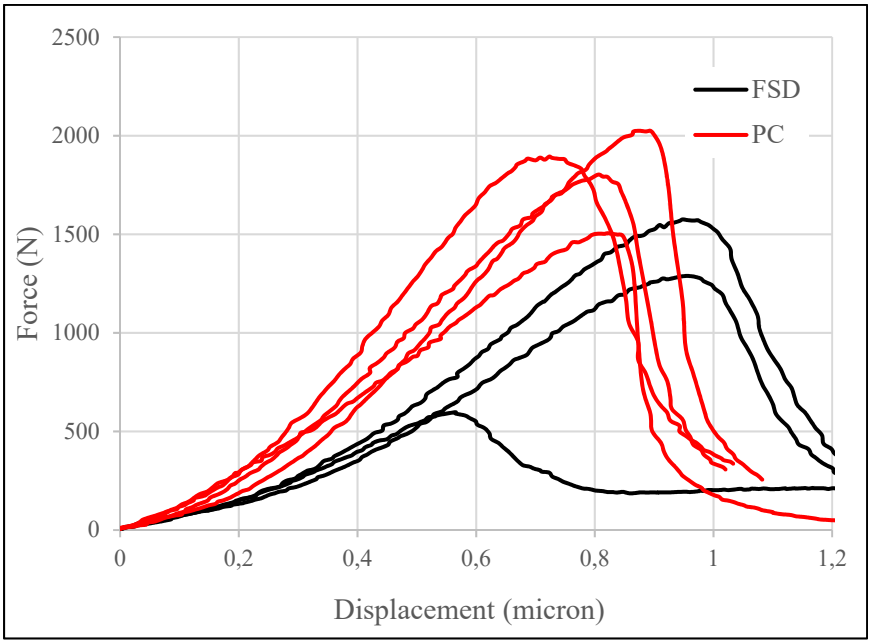

Figure 5.26 Comparison of the experimental SCB curves
for SGC compacted specimens

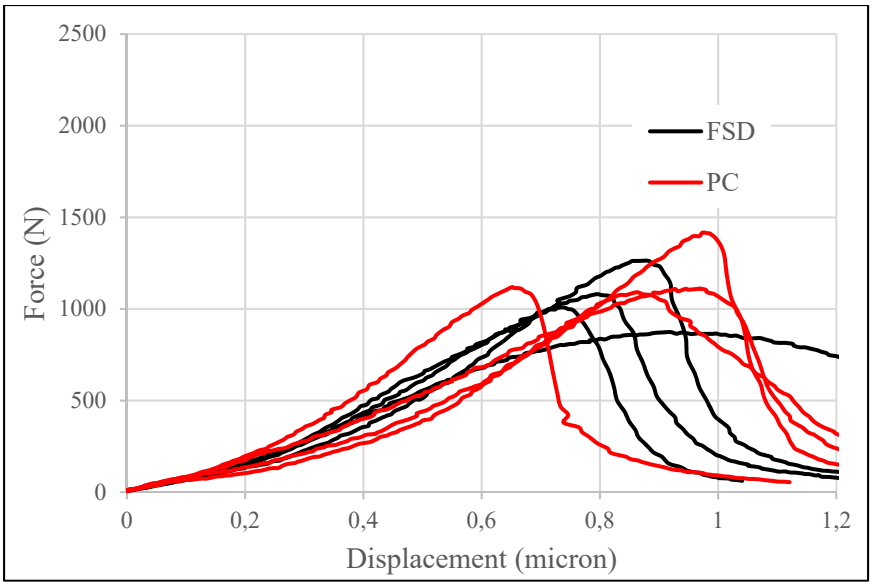

Figure 5.27 Comparison of the experimental SCB curves
for Proctor compacted specimens

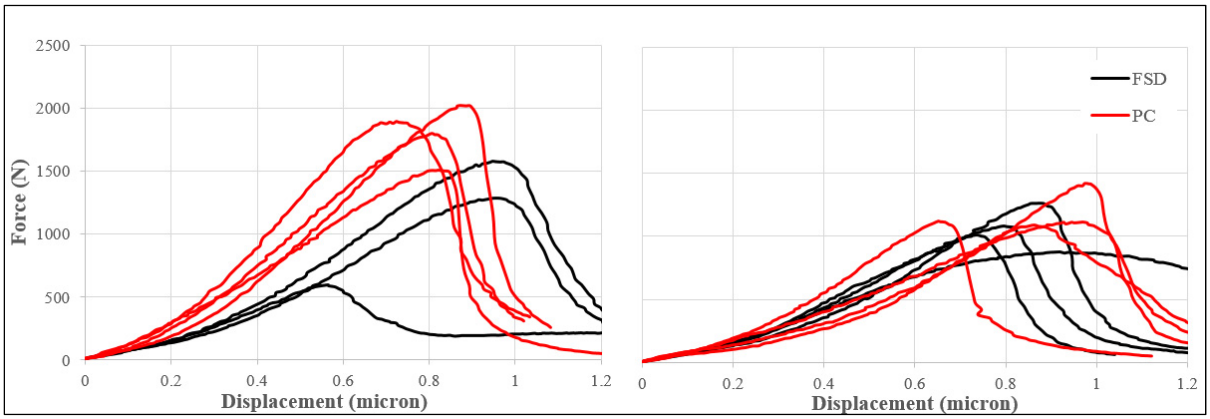

Figure 5.28 Comparison of the experimental SCB curves for SGC
and Proctor compacted specimens with different curing

CONCLUSION

The main goal of this thesis was to develop curing protocol which helps us to reflect the real condition in the laboratory. However, as it was already mentioned, there are many affecting factors which distance test results of the field specimen from laboratory made specimens. It is tried to make a comprehensive understanding by investigating multiple sources of the difference between laboratory and field specimens test results. To do so, beside different curing protocols, it took RAP sources and compaction methods as the independent variables to look at the issue more meticulously and take all into consideration. Following conclusion are drawn:

- Applying proposed curing protocol improves mechanical properties. It can be explained by saying that longer presence of water inside the specimens, which is a result of the confining the specimens, lets cement to use the water for completing its hydration. ITS, ITSM, ITR and SCB test result change with applying different curing protocols, RAP sources and compaction method.
- Confining curing condition enhances the moisture resistance of the mix. ITR test results are enhanced up to 107%.
- Controlled evaporation of the water from the specimen improves the fracture resistance characteristics of the mix. F_{max} in SCB test is about 1.5 time bigger for specimens with PC curing and SGC compaction. However, Proctor compaction loses its fracture resistance about 12%.
- RAP source is more unpredictable factor as an independent variable. ITS and ITSM result prove this fact and unlike some other criteria, RAP source act highly variable and it hampers us to characterize the material final properties according to RAP source. ITS for different RAP sources fluctuates between -5% and 33% and ITSM differences are between 2% and 54%.
- In most of the cases, as the compaction is taken as an independent variable, the main outcome is that SGC compaction eventuate in stiffer products and as it mentioned in the literature review, since it tries to simulate real field compaction, it can be deduced that SGC is more realistic laboratory compaction method and it results stiffer specimens. ITS, ITSM and SCB test approve it. However, consistency is more variable.

RECOMMENDATIONS

1. The very first recommendation is to practice the experiments in this study on two sets of specimens, actual field specimens and laboratory made specimens, which the both made by the same material and recipe. It lets us know different compactions, curing and material affects on final results which are supposed to be the same as much as possible. It casts a light on the sources of inaccuracies of reflecting the field conditions in the lab.
2. Different compactions need to be scrutinized to know the which compaction, RAP source and curing condition match better.
3. More RAP sources could be utilized in this method to establish more confident conclusion.
4. ITSM test in cooler temperatures seems so valuable since cold recycling is more affective in cold temperatures.
5. Different mix designs are another valuable subject to pursue. Applying different ingredients and/or different mix designs establish the fact that whether proposed curing protocol in this research helps to simulate the real condition and predict future condition in field or not.
6. More SCB tests and their corresponding fracture energy and fracture work calculations explain the fracture resistance characteristic for FSD and PC curing deeply.

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